Bridges, Structural Steel Work, and Mechanical Engineering Productions.

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William Arrol

BRIDGES,

STRUCTURAL STEEL WORK,

AND

MECHANICAL ENGINEERING PRODUCTIONS

BY

SIR WILLIAM ARROL AND COMPANY, LTD.,
DALMARNOCK IRON WORKS,
GLASGOW,

With Description of their Manufacturing Works, and Formulæ and Diagrams
for the Calculation of Beams, General Specifications, and other
Data Influencing the Design of Structural Work.

Partly Reprinted from "ENGINEERING."

Published for Private Circulation by
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Preface.

THIS work, mostly reprinted from "ENGINEERING," is illustrative of work undertaken by Sir William Arrol and Company, Limited, during the last forty years. It incidentally records the history of the modern developments in the Art and Science of Bridge Construction, as the undertakings successfully completed by the firm are not only representative of varied practice in bridge building, but include such historical structures as the Forth, Tower, and Tay Bridges. Many of the bridges described have been constructed from the firm's own designs.

In the design and construction of another class of structure, that of Steel Framed Workshop Buildings, the firm has also taken a leading place, and many examples of these buildings are to be seen in the large industrial establishments throughout the United Kingdom and many foreign countries.

Another branch of engineering has been developed in the design and manufacture of machine tools, hydraulic riveting machines, bending presses, cranes and pumps, shipyard crane structures and equipments, electric hammer-head cranes, and in pneumatic sinking plant for bridge foundations. Sir William Arrol and Company, Limited, have also taken a prominent part in the development of mechanical appliances for the charging and discharging of gas retorts, and in the mechanical handling and transport of coal and other materials.

The Editor is indebted to Mr. Adam Hunter, M. Inst. C.E., for the notes memoranda, and general specifications relating to the design of structural work which is published in the Appendix. The mathematical formulæ have been carefully checked and extended by Mr. Robert Boyle, B.Sc., and several new cases of inclined beams, eccentrically loaded columns and portals have been given. The specifications will be of service to Engineers in standardising the work in the Design and Manufacture of Bridges, Cranes and General Structural Steelwork, and will ensure the best practice and workmanship. The data have been prepared and compiled to afford useful information to those interested, and references are given where fuller information may be obtained.

LONDON, *July* 1909.

Contents.

List of Illustrations.

Errata.

Page 260, last line, "page 737" *should be* "page 803."

Page 287, line 19, *for* "measure x from c," *read* "measure x from C."

Page 288, line 20, *for* "measure x from c," *read* "measure x from C."

Page 299, line 5, Expression (ii) for M_x, *for* "$- w_b x^2$, *read* "$+ w_b x^2$."

Page 311, line 1, *for* "supported at B," *read* "supported at A."

Page 329, line 7, *for* "$M_B = - R_C C$," *read* "$M_B = - R_C c$."

Page 329, line 7 at side, *for* "is $= - R_0$," *read* "is $= + R_0$."

Page 372 (figure) Strength of Rings. Point C is at the centre of the thickness of the ring in a diametral plane at right angles to the links.

Page 376, line 11 from bottom, *for* "above," *read* "following."

Foundations of Success.

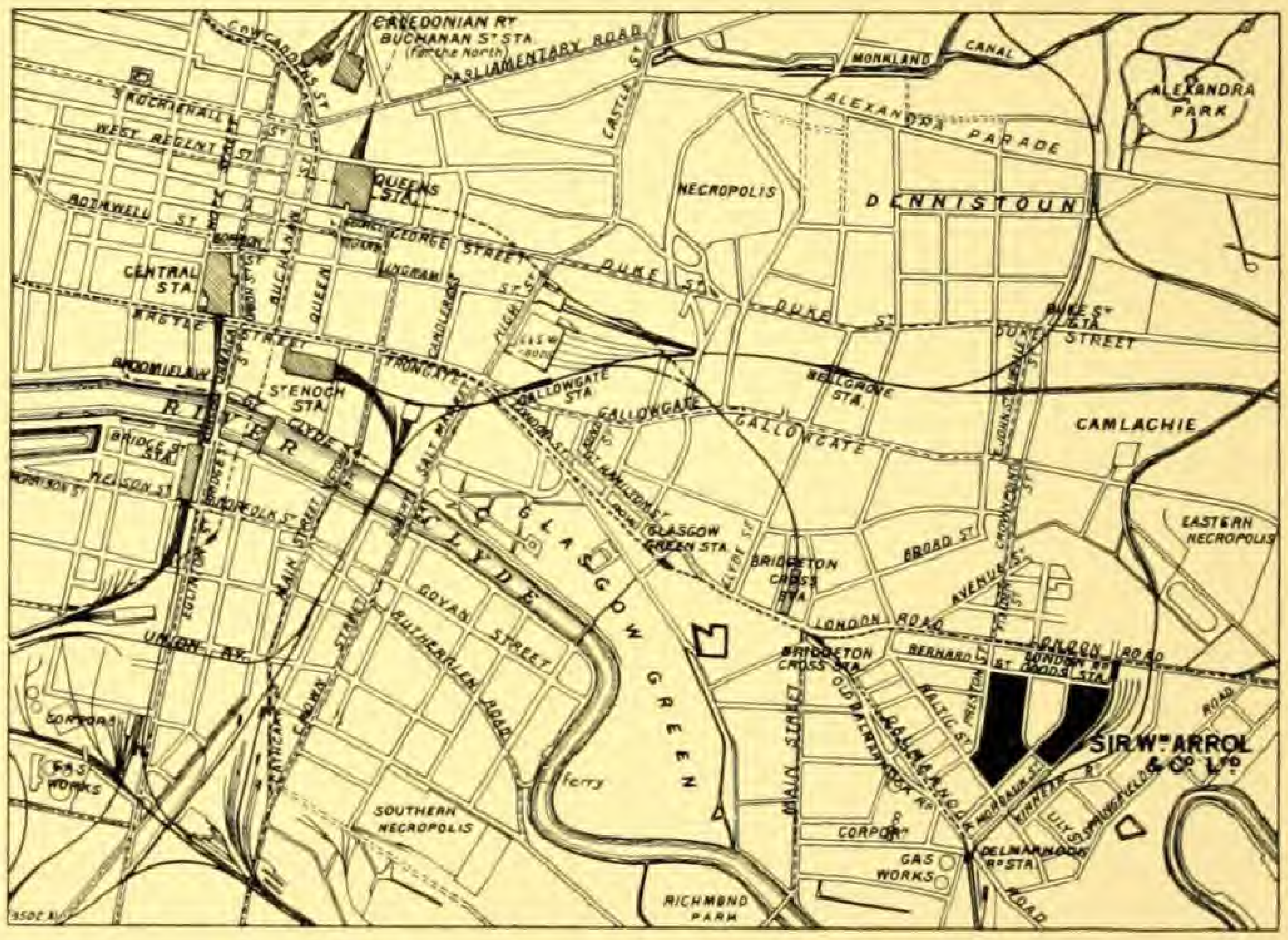
THE high position attained by Sir William Arrol and Company, Limited, is largely due to the fact that much of the work they have done has been of special difficulty and of great magnitude. It has thus made demands on them that could only be met by much practical engineering skill and by the evolution of special appliances.

As example, we may take the case of bridges—perhaps the most widely known, but not the only, branch of the Company's business. The designer may formulate the laws of stresses, and conform to them in plans and sections of piers and girders, and struts and ties; but the embodiment in material form of the most perfectly-conceived ideas requires the solution of many difficult problems by the constructor. Such work has often to be carried out in difficult situations or against adverse natural conditions. Unstable or uncertain strata and abnormal elemental forces may combine to exercise the resource and patience of the builder. Indeed, many schemes, which by virtue of their boldness are regarded as triumphs of genius, have largely depended for their success upon operations, of a more or less novel character, connected with construction and erection; and for these special methods and apparatus have had to be devised by the builder. He is thus a necessary corollary of the designer, and deserves an equal measure of credit.

The originality displayed by Sir William Arrol and his colleagues in the Company in the execution of great and difficult works has, on more than one occasion, created new precedents in engineering practice, and to this quality there has been rightly attributed the satisfactory completion of several world-famed structures. The invention, in 1875, of a machine for drilling all the component plates and angles of the booms of heavy girders, and the construction of hydraulic machinery for riveting them together with a rigidity impossible to manual effort, were typical preludes to the great achievements in connection with sub-aqueous foundations, and the erection of such structures as the Tay Bridge, the longest viaduct in Britain; the Forth Bridge, the greatest cantilever structure in the world; the Tower Bridge, with the heaviest bascule yet made; the Barrow Bridge, in Ireland, with its foundations extending 117 ft. below high-water level; the Swale Bridge, in Kent, with the heaviest rolling-lift span in England; the rebuilding of the high-level Redheugh Bridge over the Tyne at Newcastle, without interfering with traffic on the old structure; and the erection of the new Caledonian Railway Bridge over the Clyde at Glasgow, with its large foundation caissons and superstructure of 11,000 tons. These and other great undertakings which need not here be specified have successively contributed towards winning for the Company a prominent place in the world's list of constructional engineers, and have added to the engineering renown of Great Britain.

The beginning of Sir William Arrol's Works dates from 1872, when the nucleus of the present large establishment was commenced in the east end of Glasgow, amidst hay and cornfields, where now a great industrial community

surrounds the Company's extensive shops. And here it may be parenthetically stated, in order to obviate misunderstanding, that Sir William Arrol has never been associated, in name or otherwise, with any other bridge, roof, or constructional steel works in Scotland than those of Sir William Arrol and Company, Limited. The map reproduced below, showing the location of the Company's



Map Showing Position of Works in Glasgow.

Dalmarnock Works, and the tramway routes from the various railway termini in Glasgow, may assist the visitor in finding his way to the establishment.

The first work undertaken at the Dalmarnock establishment was boiler-making ; and, as indicative of the fact that, then as now, sound workmanship was the aim realised, it may be mentioned that when twelve of the first boilers made were recently removed from the works of the Steel Company of Scotland, because higher steam pressures were necessary, they were found to be in good

condition after their thirty years' service. Girder work was entered upon at an early period, and the first notable contract was for all the bridges of one of the important Edinburgh suburban railway lines. The larger spans ranged up to 60 ft. and 70 ft., to carry the railway over the water of Leith. What a contrast is presented by the 1730 ft. spans of the Forth Bridge, also in the vicinity of the capital of Scotland!

A prominent success followed in the building, in 1875, of the Bothwell Viaduct over the River Clyde, for the Hamilton and Bothwell Branch of the North British Railway. An engraving of the viaduct is reproduced on the opposite page. This work consisted of seven spans, of a total length of 727 ft. The chief interest in this case was associated with the method of projecting the continuous girders across the river, from pier to pier; this was done from one shore on rollers, actuated by ratchet-bars. This procedure is now largely adopted, but at that early stage this successful method of overcoming difficulties gained credit for the Company.

It was only appropriate, therefore, that in 1875 the Caledonian Railway should award to the Company the contract for building the heavy viaduct across the Clyde to carry their main lines of railway into the present Central Station in Glasgow. This bridge is of five spans, the largest of 200 ft., with a total width of 50 ft. The aggregate weight of iron used in the structure was 3000 tons. The girders were very heavy, there being in some cases fourteen thicknesses in the booms. As it was desired, in the interests of strength and economy of manufacture, to drill all the plates in a complete section at one operation, instead of separately punching the holes in each component plate, the Company devised a drilling-plant

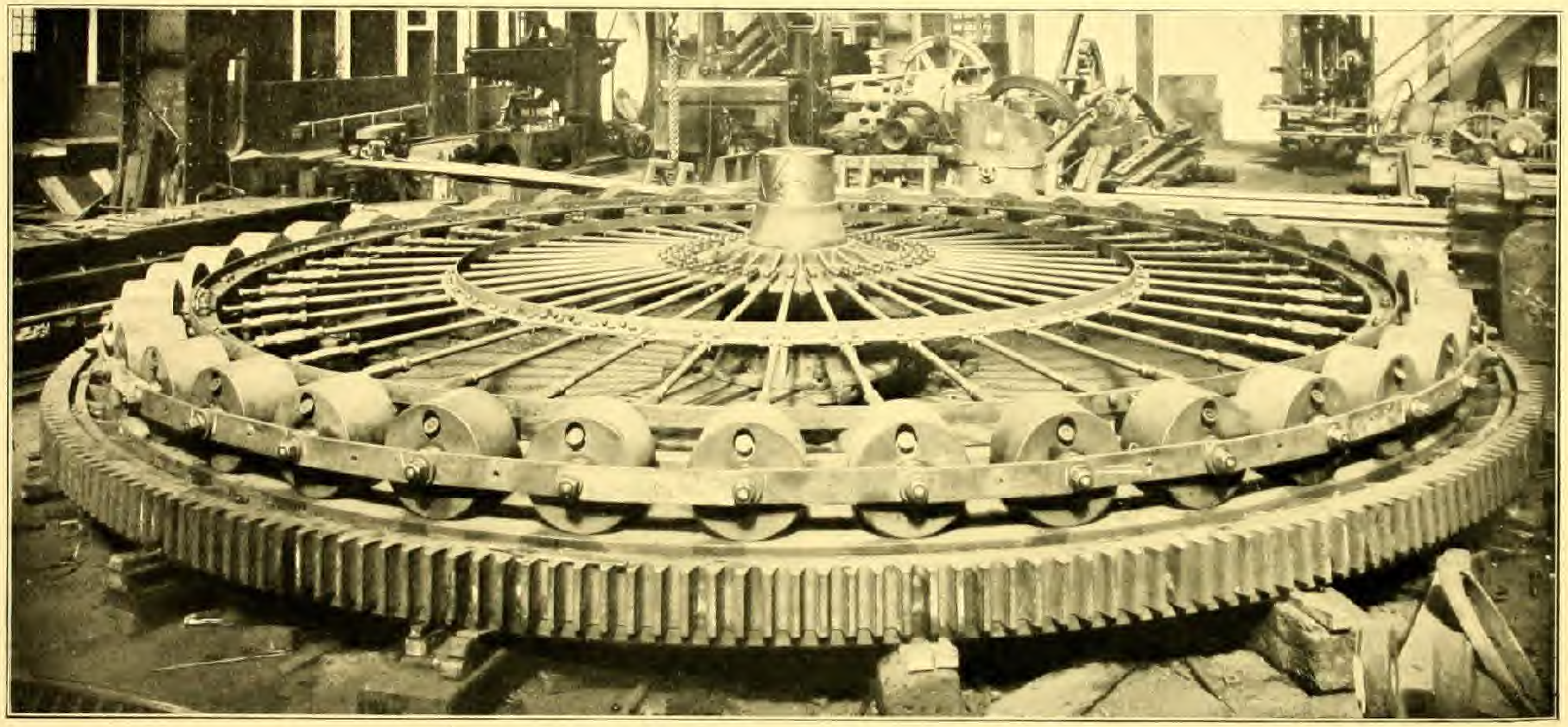
The Bothwell Viaduct.

for carrying out these operations on the girders. At the same time, hydraulic power was introduced for closing the rivets through these heavy multi-plated booms. The hydraulic pressure for closing the rivets, equal to 1000 lb. per square inch, necessitated careful research before a satisfactory flexible supply-pipe could be made; but, this difficulty overcome, the system was soon extended, and by 1880 there were many applications of hydraulic riveters in shipbuilding and boiler-making, as well as in bridge and other structural steel work.

It may be interpolated here that, in 1905, the Company raised bodily, by hydraulic jacks, to a height of 3 ft., the girders and decking of this bridge, some of the spans weighing 800 tons. This was done so that the rail level would coincide with that of the new bridge constructed alongside. The latter is 752 ft. long, averaging 120 ft. in width, and the steel work weighs 11,000 tons.

The experiments necessary for the invention of the drills and riveters, and the evolution of new forms of tools suitable for the growing variety of work, resulted in the extension of the engineering branch of the Company's Dalmarnock establishment; and, with that enterprise which has always characterised the concern, experiments on pumping plant were undertaken, so that ultimately the Company produced all the units of an hydraulic-power plant, as well as many tools for the utilisation of such power.

Several of these tools will be mentioned when we come to describe the engineering department of the works; and presently reference will be made to the important mechanical appliances connected with the sinking under compressed air of bridge, and other sub-aqueous, foundations. The extensive experience of the Company



Roller-Path for a Large Swing-Bridge.

has enabled them to effect many important improvements in air-locks.

The making of the machinery for working opening spans of bridges is another notable department. The engraving on the previous page shows the roller-path, etc., for a large swing span.

Another branch is the manufacture of hydraulic and other cranes, the designs of which have been perfected as a result of practice in their use. A kindred department is the manufacture of coal-conveying and gas-retort charging appliances, wherein hydraulic power is utilised with an efficiency begotten of long association with this prime mover.

To return to the chronological narrative, the Company in the 'seventies constructed several important bridges at home and abroad; but the first structures which call for special mention are the new Tay Bridge and the Forth Bridge, which were built simultaneously.

The contract for the Forth Bridge[1] was signed in December, 1882, and the work was finished in 1890: an achievement the merit of which is proclaimed by the fact that the six cantilevers of 680 ft., with their respective piers, and the approach viaducts and the abutments, make up an aggregate length of 8295 ft. 9½ in. Of iron and steel alone there were built into the structure over 60,000 tons. The building of the immense members forming the cantilevers, piers, etc., involved the design and construction at the Company's works of many ingenious machine-tools and appliances.[2]

The new Tay Bridge is of equal interest, not only because of its great length—10,711 ft.—but in view of the difficulties associated with the construction of the

[1] ENGINEERING, vol. xlix, page 213. [2] *Ibid.*, page 246.

The Forth Bridge.

piers. It was here that the Company introduced on a large scale the system of pontoons for sinking caissons.[1] These were made up of a series of water-tight tanks, with intervening spaces through which the cylinders of the piers were sunk. To support the pontoons, columns or legs were passed through apertures at their corners. These columns rested on the river bed, formed guide-posts, and were fitted with means whereby the height of the pontoon could be regulated. These working platforms, which were raised or lowered by hydraulic jacks, carried all the machinery and gear for the building of the piers. The cylinders were constructed in convenient lengths on the shore, conveyed to the pontoons, and raised by cranes to the working platform. On being lined with brickwork and loaded, they were lowered by hydraulic power into position. This proved quite a satisfactory procedure, and the eighty-five piers were easily and quickly completed. This bridge—Sir William Arrol and Company, Limited, were concerned only with the second structure across the Firth of Tay—necessitated the use of 27,370 tons of iron and steel, besides other material, for the superstructure. The building of this bridge, begun in 1882, was completed in 1887.

The next outstanding structure to which reference should be made, in even a brief historical retrospect of the Company's work, is the bridge over the Hawkesbury River in New South Wales.[2] The whole of the girder work was constructed by the Company. This bridge has a total length, between abutments, of 2899 ft., divided into seven spans.

The Tower Bridge,[3] commenced in 1886 and completed

[1] ENGINEERING, vol. xxxii., page 575; vol. xxxix., page 689; vol. xlii., page 664.
[2] *Ibid.*, vol. xliii., page 370.
[3] *Ibid.*, vol. lvi., pages 353, 382, 417, 448, 471; and vol. lvii., page 852.

The Tower Bridge.

in 1894, has three spans: the side ones, on the suspension principle, are 270 ft.; and the centre, the bascule span, is 200 ft. The two river piers, 70 ft. wide, accommodate the machinery for raising and lowering the bascules. They also support towers, which rise to a height of 120 ft., to carry the suspension chains and two high-level foot-passenger bridges. These upper platforms are reached by stairways in the towers, the idea being that they should be used for passenger traffic when the bridge was open for the passage of river craft. Experience, however, has shown that the bascules, although they weigh each about 350 tons, are opened and closed so quickly that the interruption to traffic across the bridge is of such short duration that passengers prefer to wait rather than use the high-level bridges. The entire steel work, aggregating 10,000 tons, was carried out by Sir William Arrol and Company.

The Company were also responsible for twelve of the largest bridges on the Manchester Ship Canal.[1] Most of these are of outstanding importance. The largest of the viaducts carry the London and North-Western Railway across the River Mersey and the ship canal near Warrington. Over each water-way there is a girder span, and on each shore a masonry arch. The spans are each 160 ft., and the girders have a total length of 173 ft., with a height at the centre of 21 ft., tapering to 14 ft. at each end. The approach arches of masonry are each of 25 ft. span. Near Partington there is another large bridge for carrying the railway of the Cheshire lines Committee over the River Mersey. There the centre span is 150 ft. with two side-spans of 103 ft. A third bridge, also for carrying a railway, has three main girders each 156 ft. long, spaced at 28 ft. centres.

[1] ENGINEERING, vol. lvii., page 114.

Of the seven large swing bridges needed for road traffic over the Canal,[1] Sir William Arrol and Company built five, and these are all of the heaviest type. In all cases the span is 120 ft., but the width varies; the weights of the swing spans range between 550 and 1350 tons. The heaviest of the bridges are those near Latchford and near Warrington, the road in these cases being 36 ft. wide. In each instance the swing span rotates on sixty rollers, on a circular-path 38 ft. 9 in. in diameter, constructed on the canal bank. Such a roller-path is illustrated on page 7. Being quickly operated, these swing bridges have practically nullified any inconvenience that might have arisen from the existence of a ship canal through a country where traffic is very extensive, owing to the adjacence of so many mines and factories. As indicative of the extent of work carried out by the Company in connection with the Manchester Ship Canal, it may be said that it required the use of about 10,000 tons of constructional steel work.

A typical Scherzer roller-lift bridge is that built by the Company in 1904-1905,[2] to carry the South-Eastern and Chatham Railway, and the public highway, over the Swale, in Kent. The lift span is 65 ft. in length, and of 520 tons in weight, including lead and iron balance weights; yet the time taken to raise it is only 50 seconds, and the power required 9 brake horse-power.

Many other bridges might be similarly described, but we content ourselves in this historical review with a reference to four more structures: one over the River Tyne, another over the River Wear, and two viaducts in Ireland.

The first-named, built in 1900-1901, at Redheugh, is

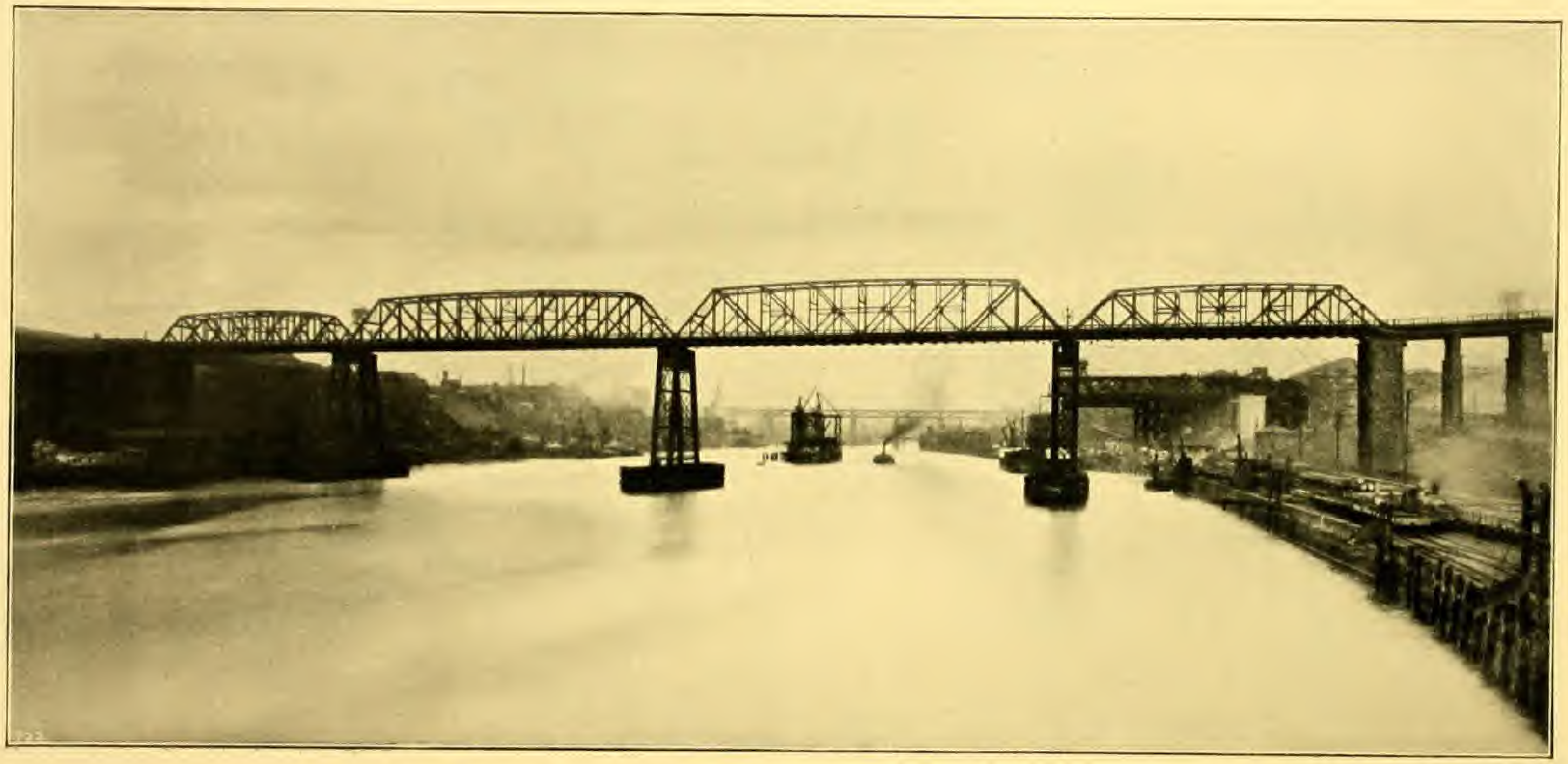
[1] ENGINEERING, vol. lvii., page 118.

[2] *Ibid.*, vol. lxxix., page 762.

for road traffic between Newcastle and Gateshead. The old structure had, in the first instance, to be supported on timber trestles carried from staging around the old piers, so that the new piers could be sunk on the same site. Each of the three new main piers consists of four cylindrical caissons, sunk to a depth of 65 ft. below high-water level. Into the four spans—two over the channel of the river, each of 248 ft. in length, and two from the shores of 169 ft. in length—there was worked 2750 tons of steel. They were built from the new piers cantileverwise. The roadway is 23 ft. wide, with footpaths on the outside of the lines of main girders. The main members of the new bridge were erected parallel to the old girders: one of them on a footpath, and the other outside of the old girder. They were at a slightly higher level than their original seating, so as to interfere as little as possible with the old bridge. The transverse-girders having been constructed in trenches across the roadway, the main girders were lowered and the new decking built. The old spans were then removed, and finally the new bridge was moved laterally into its correct position, taking the line formerly occupied by the old structure.[1]

The Wear Bridge is in course of construction at Sunderland, and is designed for two decks; the upper for the double line of railway, and the lower for the roadway, with footpaths outside of the girders. The height will give a clear headway above high-water level of 85 ft. The total length of the bridge and approaches will be 1560 ft.—the river span is 330 ft. long—so that the girder steel work is very heavy, totalling 9000 tons. The method proposed for placing the heavy central girder in position is ingenious, and suggestive of the resource of the Company.

[1] ENGINEERING, vol. lxxii., pages 550, 644.



The Redheugh Bridge over the Tyne at Newcastle.

The shore spans having been completed, towers are to be erected upon the river piers, the shore girders being utilised for anchorage. The girders for the river span will be constructed from each pier as overhanging members, and will be supported during the process of erection by ties from the tops of the towers. When the halves thus constructed meet in the centre and are joined, the temporary supports will be removed. Here, as in many of the modern bridges, chief interest was associated with the foundations. One of the piers necessitated the sinking to a depth of 75 ft. under high-water level of a rectangular steel caisson 63 ft. long and 35 ft. wide. This operation has been carried out under air-pressure in a satisfactory way.

The bridges over the rivers Barrow and Suir,[1] built in 1905-1906, are also notable for the great depth of foundations as well as for their length. The Barrow Bridge is the longest railway structure in Ireland, being 2131 ft. long between the faces of abutments, made up of thirteen spans each of 148 ft. in the clear, with a swing span over the channel, giving a passage 80 ft. wide for the traffic on each side of the centre dolphin. The Suir Bridge is 1205 ft. long, and in this case the opening span is of the Scherzer roller-lift type. The piers of the Barrow Bridge are founded in many cases at a depth below high-water level of from 100 ft. to 117 ft., the latter the greatest depth yet reached in compressed-air work for bridge foundations in the United Kingdom.[2] The maximum pressure was 43.5 lb. per square inch above atmosphere.

The work was successfully carried through at this great depth, and under this high pressure. This is as

[1] ENGINEERING, vol. lxxxi., pages 673, 716, 780, 841.

[2] Probably the greatest depth reached for bridge foundations was in connection with the Hawkesbury Bridge in New South Wales, namely, 162 ft. below high-water level, but in this case the work was not done under compressed air.

one would expect, in view of the great experience of the Company in such work; no British firm has carried out such extensive operations in this department of engineering.

Although subaqueous foundations were formed in the later years of the eighteenth century by means of diving bells, the use of compressed air for excluding water from working chambers in caissons dates only from the latter half of the nineteenth century, and the first application for important work was probably in connection with the building of the St. Louis Bridge in 1870, where the deepest foundations are 108 ft. below water level.[1]

The success of the process in the sinking of the caissons of 70 ft. diameter for the piers of the Forth Bridge, in 1884, established the practice in this country, and since then it has become very extensive. In 1876 Sir William Arrol and Company carried the piers for the Caledonian Railway Bridge across the Clyde at Glasgow to a depth of 70 ft. below high-water level, and since then they have worked down to 85 ft. in the same river for the piers of the new railway bridge alongside the old structure, and also for the founding of quay walls for Glasgow Harbour. They worked to a depth of 65 ft. in the Tyne for the piers of the Redheugh Bridge, and to a similar depth in connection with the Swale Bridge; while on the Wear, as we have already stated, their operations extended to a depth of 75 ft. below high-water mark. Through the fine sand of the Nile they worked to 75 ft. below Low Nile, so that the work on the Irish river is only the natural development of former achievements.

It will be recognised that compressed air offers an

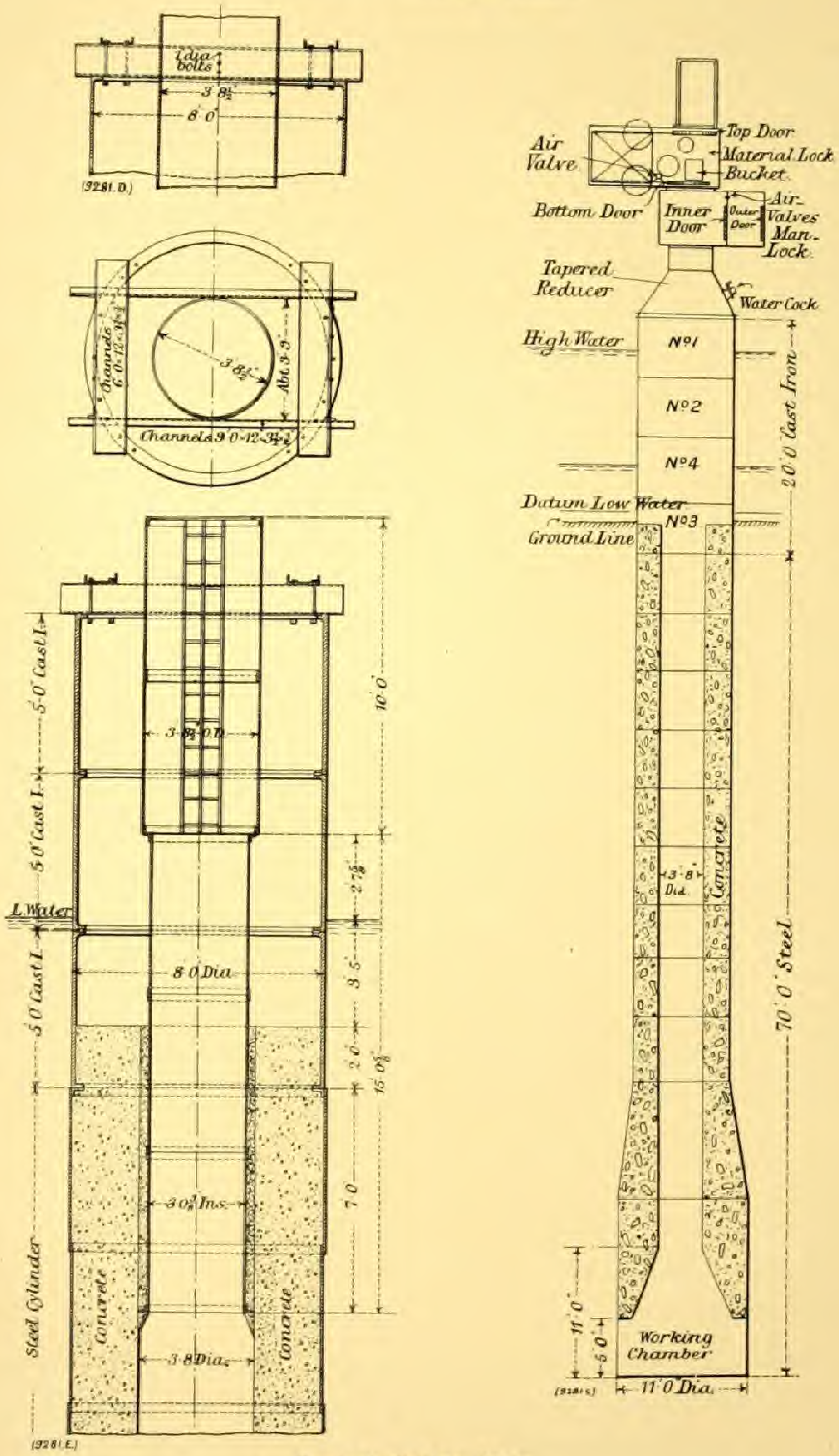
[1] See "Sub-Aqueous Foundations, with Recent Examples of the Use of Compressed Air," by J. E. Tuit, M. Inst. C.E.

enormous advantage, because in the event of the cylinder meeting an obstruction in the process of sinking, such as boulders or sunken barges—as, indeed, was experienced in the Nile—there is no danger, and but little delay to the progress of the work.

Timber is extensively used in the United States of America for compressed-air caissons, but Sir William Arrol and Company prefer steel, which, although perhaps more costly, and involving some delay in the preliminary stages, enables operations to be carried forward with greater surety, and with more likelihood of overcoming the disadvantages of such obstructions as those referred to.

The steel usually extends up to within 10 ft. or 12 ft. of the bed of the river, the walls for the remaining height, where subjected to the action of water, being of cast iron. Usually, a lining of concrete is put in during the process of sinking, as illustrated in the case of the typical bridge pier on the opposite page. This illustrates a pier of the Suir Bridge, already referred to. The most recent practice has been to form in the inside of the concrete lining a steel casing as shown, so that the cast iron is not subjected to any pressure from within. A temporary shaft of steel is thus continued to the top, and the air-locks are carried upon it. In order to distribute the weight of the temporary shaft and the locks, etc., girders are constructed on the top of the cast-iron cylinder, and secured by bolts to it. The inner shaft is suspended to these girders, while the locks are superposed upon them.

The details of the air-locks have been evolved from long experience, and the arrangement is well shown on page 21. These views illustrate the work of sinking foundations for the piers for a 150-ton jib-crane at the dock of the Clydebank Shipbuilding Yard of Messrs.



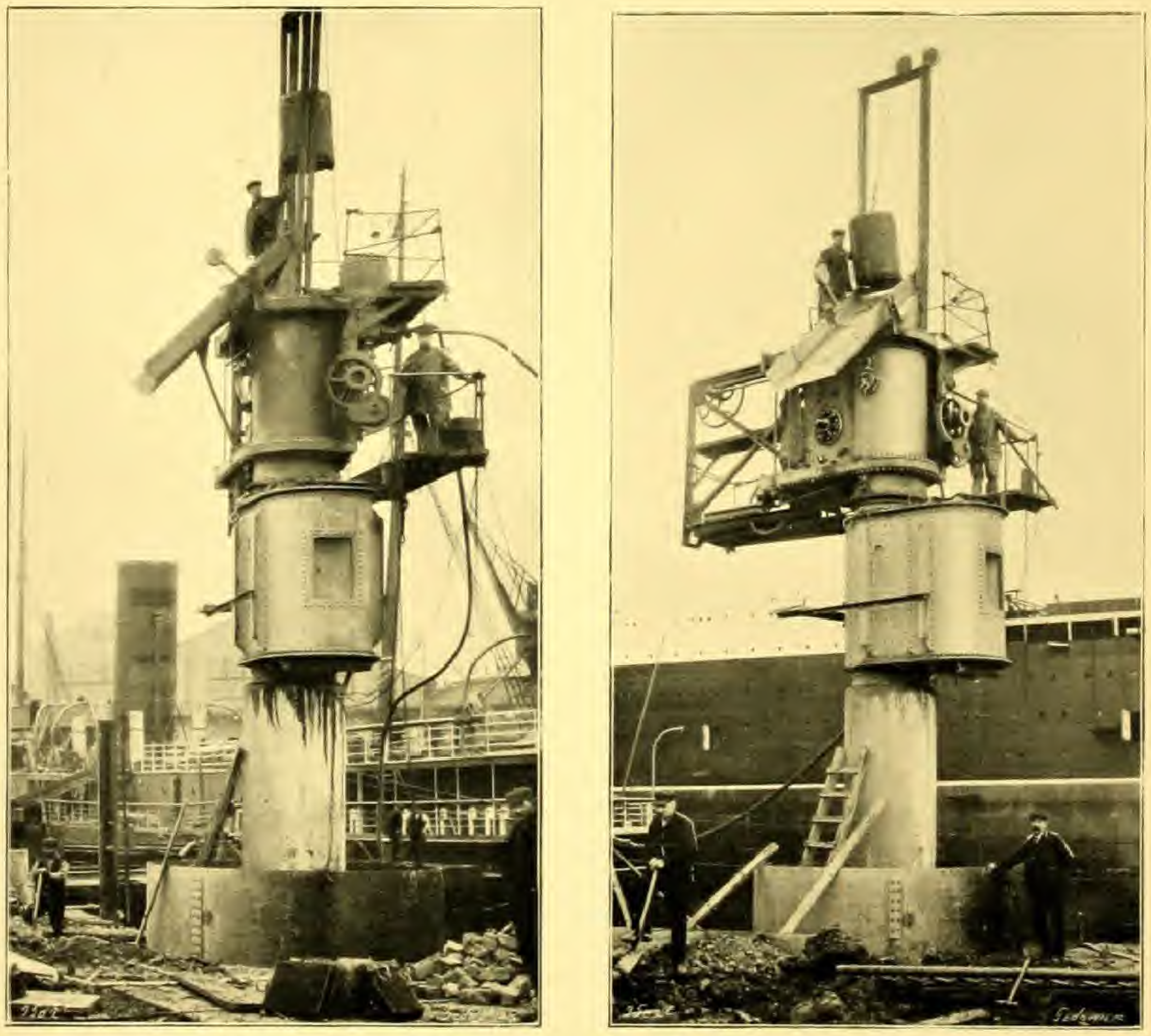
Typical Bridge Piers.

John Brown and Co., Ltd. The man-lock is placed next the cylinder, and the material-lock above it, with an overhead gear for raising the buckets out of the lock. The man-lock consists of two chambers; the material-lock is on the same principle, but has horizontal doors. The doors are fitted with locking-gear, so that when a full bucket is raised, the lower compartment is closed to the cylinder before the bucket can be passed to the upper compartment. The bucket travels through the man-lock, but the compartment for the ingress and egress of the workmen is entirely independent of the shaft through which the material passes. The doors are manipulated by rack-and-pinion gear, actuated by a hand-wheel. The buckets are raised and lowered by steam-winches placed outside, the shaft to the winding-drum inside passing through an air-tight gland on the side of the material-lock.

Experience has also enabled the Company to formulate definite regulations, so as to minimise the chances of the men suffering from compressed-air illness. So long as the pressure in the working chamber is not above 30 lb., the men are permitted to work eight hours per day, but when this pressure is exceeded, the day's work is limited to six hours or less, in both cases in two shifts, with an intervening period of about two hours. This working time of course includes the period of passing from the atmosphere to the high pressure, and *vice versâ*. With high pressure the period prescribed is twenty minutes for the former, and twenty-five to thirty-five minutes for the latter, passage. As soon as the men are out of the lock, hot coffee is supplied, and they are advised not to go immediately into the cold air. An effort is made to secure workmen who have lived temperately, as it is

found that they are least affected by the sudden changes in pressure.

The bridges which we have described were designed



Air Locks Used in Sinking Cylinders under Compressed Air.

by eminent engineers; but, in the interval, Sir William Arrol and Company had developed a designing department which has been a large factor in their continued success as an industrial concern. The reason, perhaps, is that it has been found conducive not only to efficiency, but also to expeditious work, that structures should be designed

to conform to standard machine-shop practice, as well as with due regard to strength; and to this is doubtless attributable the adoption of the Company's design in many instances.

Some typical examples of bridges designed, as well as built, by the Company are given in the Table in a subsequent page; and here reference need only be made to a large series of bridges constructed within the past two years for the Soudan Railway, involving the use of 4160 tons of steel, and ranging in spans from 105 ft. to 55 ft.

The most prominent success of the Company's designs was in the International competition for the plans of three bridges over the River Nile at Cairo.[1] The firms participating in this competition were representative of the best bridge-builders in practically every country in the world, and the Arrol design was accepted largely because of its merit, particularly in the details of foundation work, the symmetrical proportions of girders and fascia work, and the conformity of the decorative features with Egyptian art. The largest of these bridges, which are now in course of construction, is 1735 ft. long between abutments, in 11 spans, with a centre swing portion affording two clear passages of 84 ft. in width for river traffic.

There is another department of work to which we may now refer, namely, the construction of stations and workshops. In this branch of engineering the Company have attained a success, alike in design and construction, as pronounced as in the case of bridges.

The conditions in the workshops of this country have been almost revolutionised by the construction of light steel roof-principals with extensive glazing. The first roof built by the Company was that made in 1887 for

[1] See ENGINEERING, vol. lxxvii., page 682; vol. lxxxi., page 41.

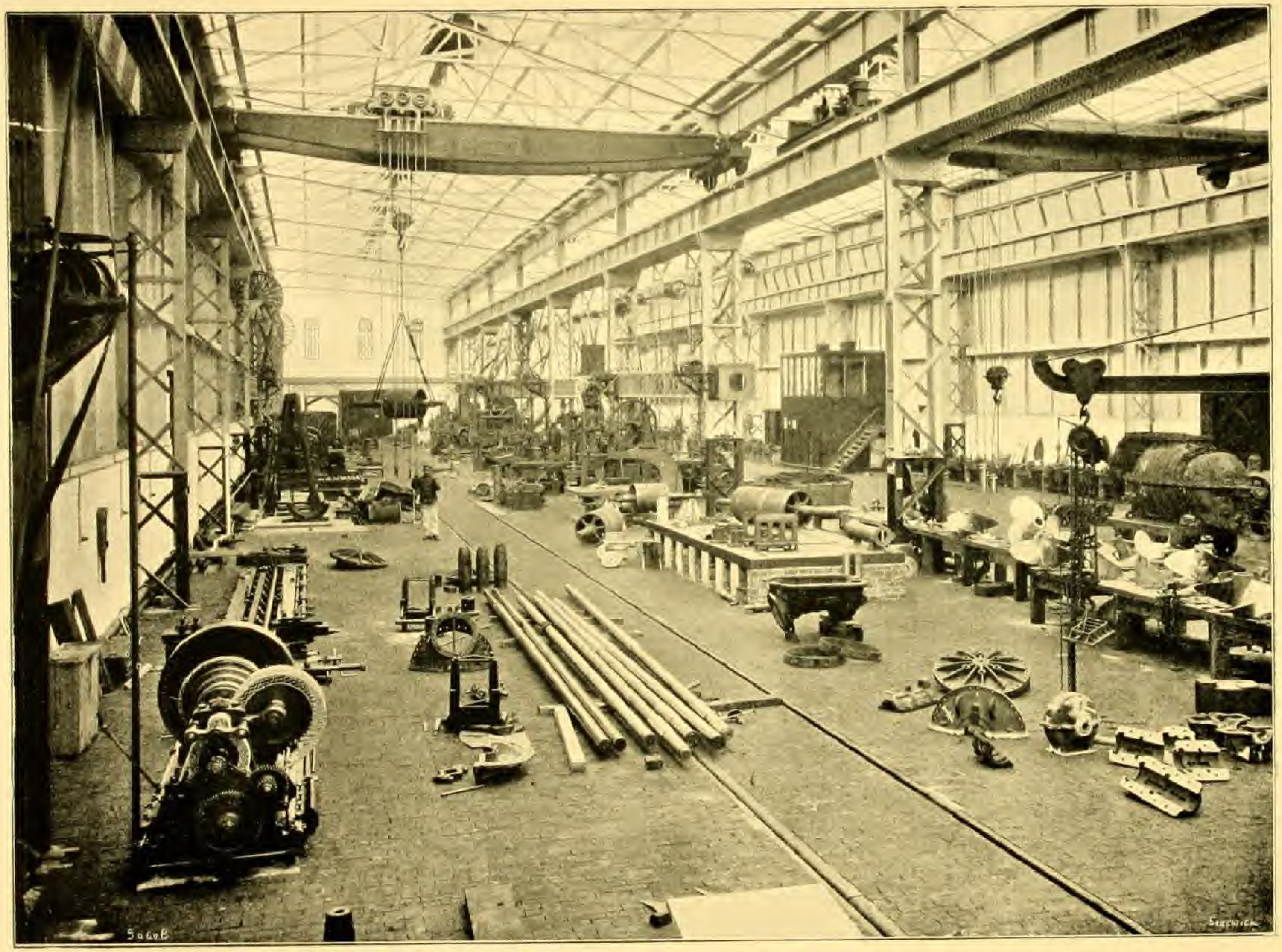
The Naval Engine Works of Messrs. William Beardmore and Co., Limited, at Dalmuir.

the smithy of Dubs' Locomotive Works in Glasgow, now owned by the North British Locomotive Company. Glazing was here introduced for the first time extensively, but the condition was insisted on that the glass should be coloured, so as to subdue the sunlight. Since then opinion has greatly changed, and one of the most prominent instances of the extensive adoption of glazing is to be found in the roof of the new engine and boiler shops at the Naval Construction Works, at Dalmuir, of Messrs. William Beardmore and Co., Ltd., illustrated on page 23. The area of glazing is nearly 3½ acres in extent. In the interval the Company have constructed a great number of factories and representative works.

The illustration on page 25 shows the roof made in 1897 for the Hon. C. A. Parsons' Works. This shop is of double interest, for, apart from the general success of the structure, it is the birthplace of the marine steam turbine: one of the most outstanding departures in engineering, which promises to revolutionise the propulsion of ships as well as the driving of electrical machinery. A list of the principal workshops erected is given in a subsequent page.

Amongst naval firms who have had buildings designed and constructed by the Company are Messrs. Vickers Sons and Maxim; Sir W. G. Armstrong, Whitworth and Co.; John Brown and Co.; William Beardmore and Co.; the Fairfield Shipbuilding and Engineering Company; Cammell, Laird and Co.; Scotts' Shipbuilding and Engineering Company; the Wallsend Slipway and Engineering Company; and Yarrow and Co.

Among general engineering firms note may be made of Messrs. D. Rowan and Co., of Glasgow; Babcock and Wilcox; Stewarts and Lloyds; the Glenfield and Kennedy



Parsons' Marine Steam Turbine Works.

Company, Kilmarnock; Marshall, Sons and Co., of Gainsborough, and others. The Charing Cross and City Electric Company of London, the Metropolitan Electric Company, and the Glasgow Corporation, are amongst the owners of electric power stations built by the Company; while there are such miscellaneous factories as Guinness' brewery at Dublin, a tannery at Canterbury, the Newburn Steel Works, Mc-Farlane, Lang and Co.'s biscuit factory, and H. and J. Templeton's carpet factory in Glasgow.

The
Dalmarnock Works at Glasgow.

The General Offices of the Company at the Dalmarnock Works, Glasgow.

The Dalmarnock Works at Glasgow.

THE Dalmarnock Works of Sir William Arrol and Company, Limited, where the constructional steel work for bridges, workshops, railway and electric-power stations, and the like, is manufactured, cover an area of some 17 acres, and the equipment includes many specially-designed machine tools. Here close upon 2000 men are employed; but this does not represent the total number of the Company's employés, as on the sites where bridges, etc., are, from time to time, being erected there are large staffs of engineers and workmen, the total usually running into many thousands. We are here, however, concerned only with the organisation and equipment of the Glasgow establishment, and the influence these qualities have upon the accuracy, economy, and rapid production which are the desiderata in all factories.

The principles which are prominently kept in view are (1) the adaptation of design, as far as possible, to suit special tools and systems of manufacture; (2) the preparation of full and clear detail drawings for the shops; and (3) the extensive use of templates for all units to ensure absolute precision. These result in the component parts being so accurate, when they enter the machine-shop and erecting departments, as to minimise time and trouble, and to eliminate the possibilities of error. In the appreciation of the great importance of attention to such preliminary detail in technical and

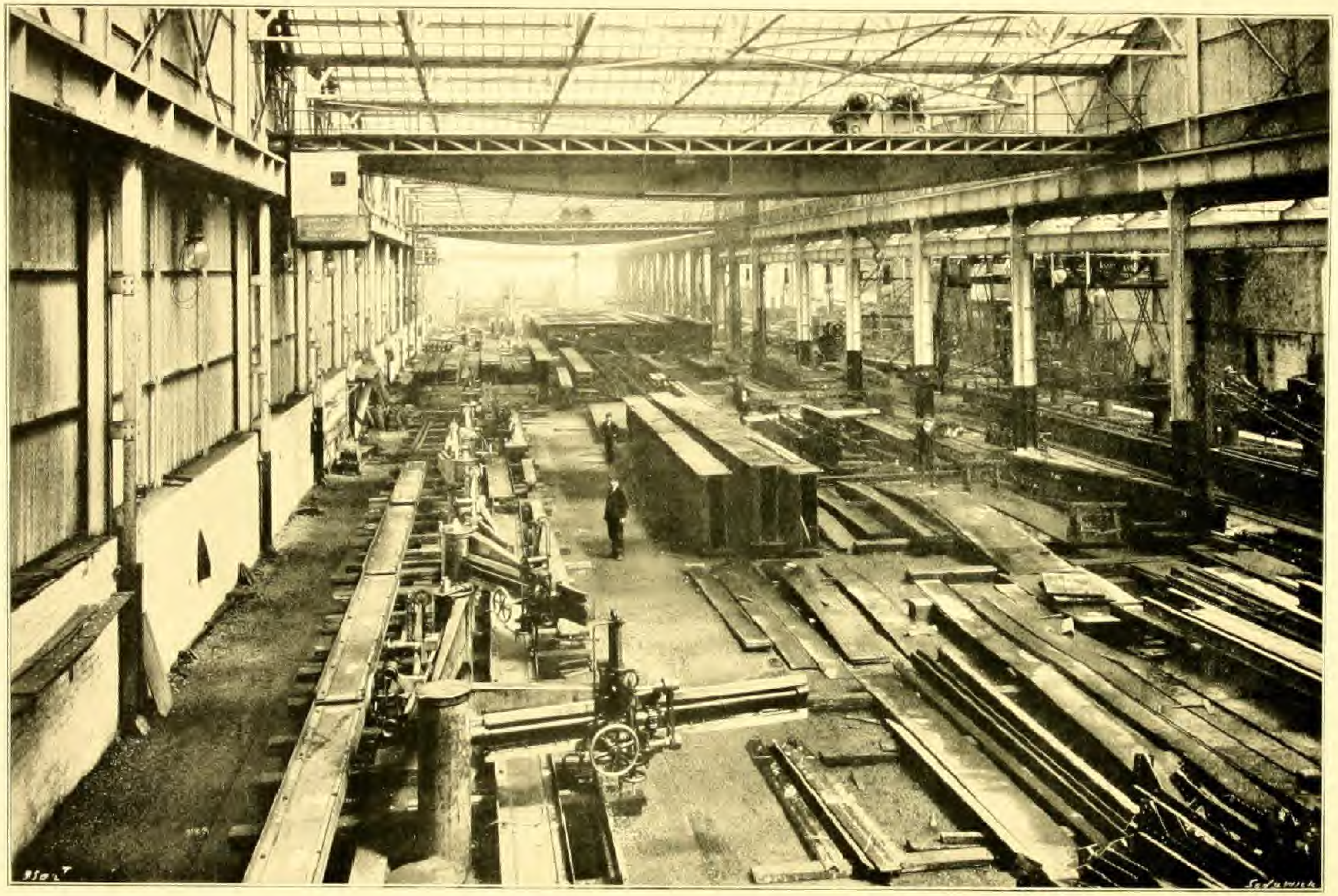
commercial matters one recognises the great experience of the founder of the Works, Sir William Arrol, and of his co-Directors, Mr. A. S. Biggart, Mr. Thomas Arrol, and Mr. John Hunter.

The Works include five erecting departments, besides the pattern, joinery, and template shops, which are common to all. Each of the five is independent, with its own

A Receiving Yard for Material.

furnaces, angle-, beam-, and plate-straightening machines, cutting and planing tools, drills, presses and riveters, so that each can complete girders to conform to plan. The view on this page shows the receiving yard for one of these erecting departments. It is 122 ft. wide, and is traversed by an overhead electric crane with a lifting capacity of ten tons. A view of the main erecting shop is given on the opposite page.

As indicative of the sequence of operations and of

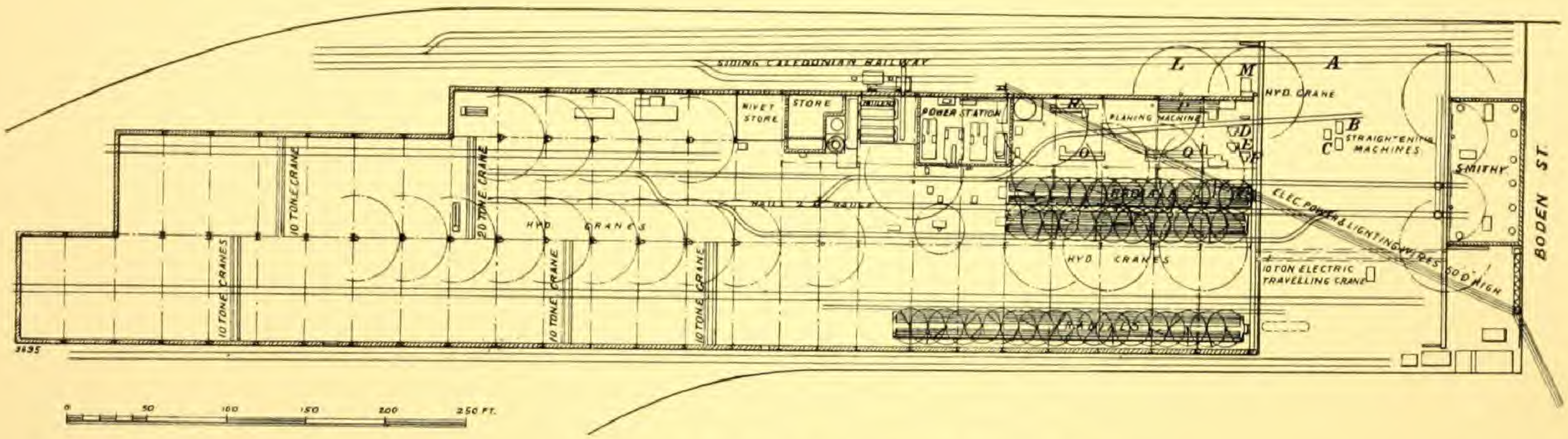


One of the Machine and Erecting Departments.

the methodical arrangement of tools, we produce on the opposite page a plan of the same machine and erecting department. The receiving department shown on page 30 is at one end. The angles and beams are discharged at A, and, when required, are passed to the straightening machines B and C, which are actuated by a 10 horse-power two-pole shunt-wound motor. In line with these are three saws marked D, E, and F, which cut the angles and beams to the required length.

These are conveyed on wagons on a 2-ft gauge railway to the benches at the drilling-machines, where they are brought into contact with the cover, web and other plates, to form girders or columns. The plates are discharged about the point marked L, and are flattened on the rolling-machine M, which takes plates up to 7 ft. in width. This tool is commanded by one of the many 3-ton hydraulic jib-cranes installed throughout the Works. The plates are replaced on wagons on the narrow-gauge railway for conveyance to the edge-planing machines marked P, Q, R, and O. Thence they pass to the benches at the drilling-machines, to be fitted together, according to template, before being drilled. Having been drilled, they are passed along the trolley lines to the erecting and riveting shop.

There is thus no retrogressive movement, no unnecessary handling or transit. To ensure these conditions, such tools as hydraulic angle-shears, presses for stamping knees, bracket and angle stiffeners, plate-joggling presses, etc., utilised at various stages of the process of machining, are dotted throughout the works, so that they stand where required in the line of progression of the component units towards the erecting and riveting shop, where the final assemblage takes place.



Plan of the Principal Girder-Machining and Erecting Shop.

To Illustrate the Sequence of Operations and the Arrangement of Machine Tools used in the Construction of a Girder.

Facility in handling is an equally important consideration in the economy of all works, and in the Dalmarnock establishment it has had adequate attention. Most of the views which we publish on succeeding pages indicate clearly that there is practically no square foot of workshop or erecting yard which is not commanded by hydraulic jib, or electric or steam travelling-cranes, and in most cases there are two.

In the design of the 120 jib-cranes installed, the Company have profited by their extensive experience. Of the number, seventy-four are of the hydraulic type. These are, as a rule, of from two to three tons lifting capacity, with a range of vertical lift of 7 ft. The hoisting time when loaded is twenty seconds, and when unloaded twelve seconds. In many cases the jibs describe a complete circle; in others the arc has been arranged to suit the situation or requirements. In addition, there are seven fixed steam cranes ranging up to 15 tons capacity, and seven locomotive cranes of 5 tons lifting capacity working on the broad-gauge railway. Standard gauge and 2 ft. gauge railways are laid throughout the works.

Instead of describing the departments successively in itinerary sequence, with their duplication of shops varying only slightly in equipment and in the details of the machine tools, we propose to follow a typical production through the various stages of manufacture, from its design to its completion. The processes differ little for all steel work, whether the ultimate form is a girder for a bridge, a caisson for forming foundations, a steel-framed structure for a workshop, station, or other building, framing for cranes, piers, or gantries, a hopper or conveyor for coal or other material, a tank or pipes. The same tools, with slight modifications, are utilised.

The designs, whether by outside engineers or by the Company's staff, are passed into the Works Drawing Office illustrated on this page. This is a well-equipped department with about thirty draughtsmen; and here working drawings are prepared, the originals being subsequently

One of the Drawing Offices.

catalogued, and arranged methodically in a large fireproof safe for future reference. As the Company are frequently called upon to alter or add to structures some years after they have been built, these stored drawings have proved very useful. The drawings are "traced," and from the tracings there are made "sun-prints" in an extensive photographic department, and these ultimately become the shop drawings. There are separate staffs for bridge work,

constructional steel work, and for mechanical engineering, with designing offices for each.

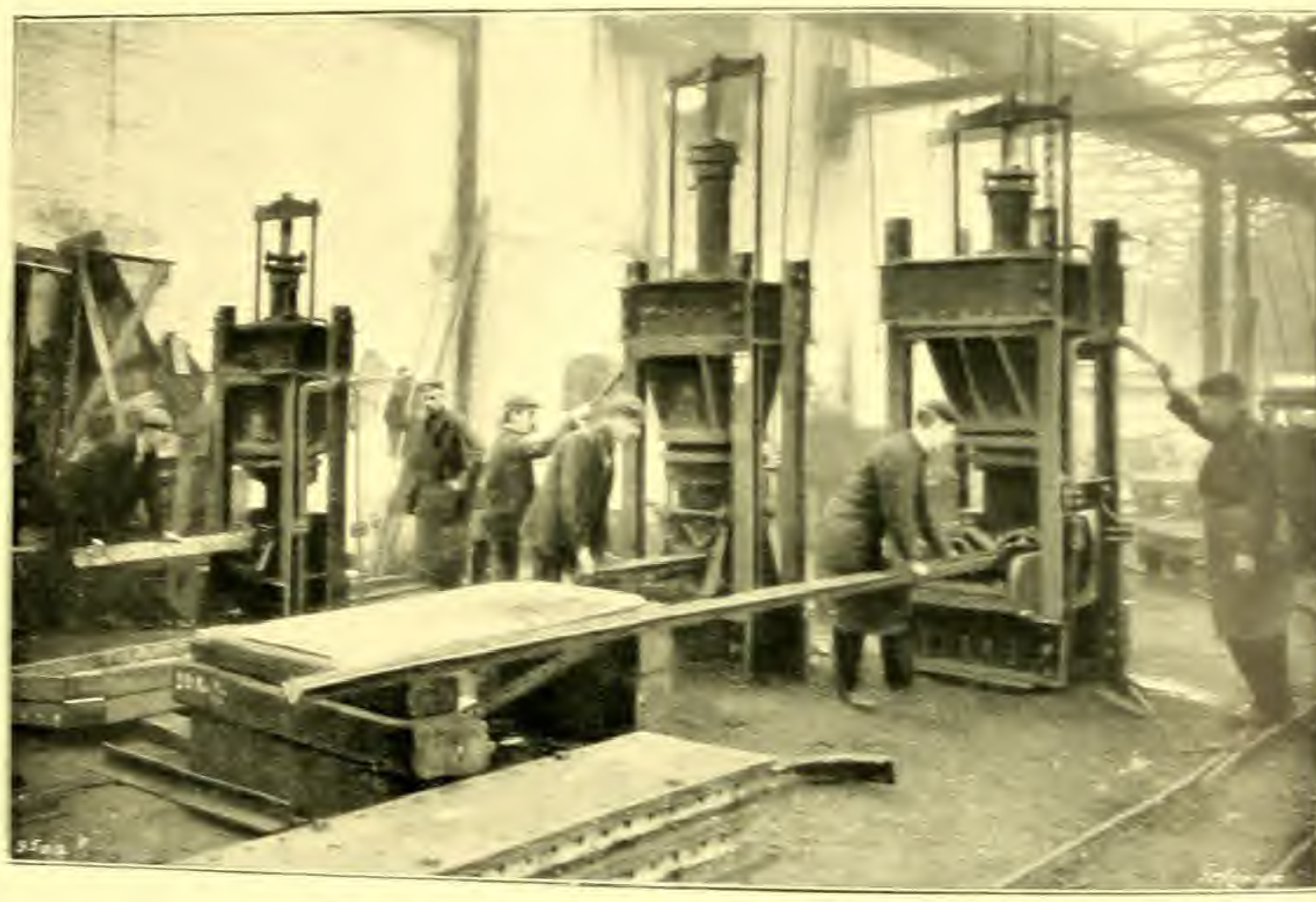
Prints and specifications are issued to the template shop, and to all the other departments concerned; nothing is left to the judgment of the worker; the rule is enforced that prescribed details shall be followed. In each case exact dimensions are specified, with a note of the nature of the work to be performed on each unit, and an indication of the marks which each must bear to facilitate erection on the site.

The procedure in the template shop, where practical work begins, makes or mars the success of the job. The efficient realisation of the designer's ideas is dependent on the precision practised here. Accuracy also influences the economy and expedition with which work is subsequently accomplished. The methods followed at Sir William Arrol and Company's Works are therefore interesting. Their main template shop, illustrated on the opposite page, is a large and well-lighted loft, 310 ft. long and 42 ft. wide, with an absolutely level and blackened floor, free from obstructions: the necessary wood-working machinery for making the templates, etc., is arranged in a separate shop on the same floor.

From the "sun-prints" the template makers draw the girder, or column, or other piece of work, to its full dimensions upon the floor, giving, of course, any camber that may be specified. The templates are formed to the same size, and are sprigged down over the lines drawn on the floor. Several units, which fit into or overlap each other in the finished structure, are thus fitted together in their flat template form, and the necessary rivet- or bolt-holes are carefully drilled in the template. The result is an exact representation of the girder or column to be made.

The Making of Templates.

The view on the preceding page of the template shop shows, built up on the floor, the templates of a span and a-half of the girders of the larger of the three bridges over the Nile at Cairo—1735 ft. long. These girders are, it will be seen, of the lattice type, and the length of girder in the illustration is nearly 300 ft.



Cutting Bars and Angles for Girders.

With such templates, and the drawings and specifications, the constructors have a relatively simple task. The bars to form the girders are flattened in hydraulic machines, and the plates are straightened in rolls. The bars are cut to the required length in hydraulic shears, illustrated on this page; but where the ends are to butt closely, as in the main booms of girders, they are sawn to ensure a finer edge.

Contemporaneously the plates have their edges planed, and there are several very heavy tools in the works for

this purpose. They are, as a rule, side and end machines, so that one side and an end may be planed simultaneously. Several of these tools thus work plates 36 ft. long and 7 ft. wide. To expedite matters, the tools have in all cases hydraulically-operated jacks for holding down the plates on the tables. These fix the plates much quicker

Planing the Edges of Plates for Girders.

than the old screw-down jacks. Two of these magnificent tools are illustrated on this page.

While this work is going forward, there is proceeding in one or other of the forges the stamping of knee-bars, curved or twisted bars, bent plates, and other small parts, to form stiffening pieces, connections, and other parts of girders.

Formerly the cutting, setting, and welding of these units to the required form was done by hand, which entailed a large amount of labour by smiths. But special tools were

devised by the Company, by which each of these pieces—of great variety of shape and size—is formed at one or two operations, and with greater accuracy of dimension than by hand-work. Such a machine consists of an hydraulic cylinder mounted horizontally on a table. On the ram-head there are mounted "former" blocks, while on the table in front there are secured corresponding dies. The red-hot bar is placed on the table between the blocks, and an hydraulic pressure of 850 lb. per square inch on the ram forms the bar between the blocks to the exact shape required.

On the opposite page there are two engravings of this process, one view showing the straight red-hot bar placed in position, with many parts completed in the foreground; in the other illustration there are shown various forms of the dies used. Not only is the operation expeditiously executed, but there is no uncertainty of weld. The whole of the metal in the bar is retained in the inside of the knee, where it becomes thicker and broader, materially adding to its strength.

While moulds or blocks can be made to suit any form, there is an advantage, in the designing of details of structures, in utilising existing specialised tools to the fullest extent. This is only one of many reasons which might be adduced in favour of the design of details being largely left to the manufacturer.

The web and cover plates, angles, knees, stiffening-pieces, etc., thus separately prepared in tools adjacent to each other, are conveyed to the table of a long battery of radial drills. In each of the yards there are such collections of drills, there being about one hundred in the Works. On the illustration on page 31 there is shown a series of these drills in two rows, the tables being of sufficient length to enable very long girders to be set out, and the

Hydraulic Machine for Forming Knee-Bars, etc.

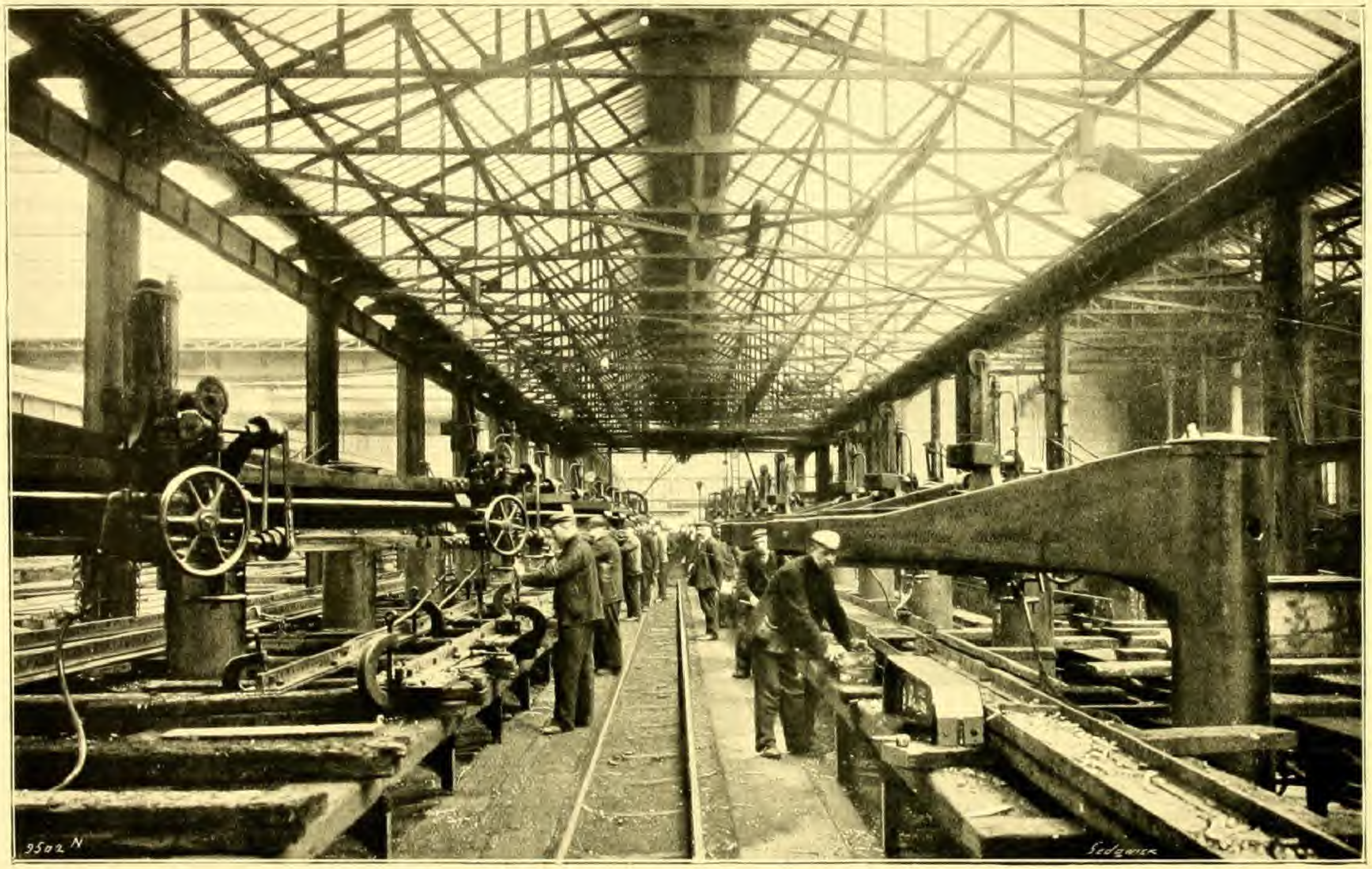
Machine and Dies for Forming Small Units of Girders.

units built up in their relative positions. The view on the opposite page shows another battery of drills, twenty-four in number, in two rows, to work girders up to 225 ft. in length at one setting.

The templates are here brought together, and the metal parts take their allotted places, when all holes are marked off. The radial arms of the drills command the whole length and depth of the embryo girder as it lies horizontally on the bench. The arms are at 12 ft. $1\frac{1}{2}$ in. centres, and have a radius of 9 ft. 6 in. Spiral drills are used throughout; and as a consequence of careful tests of tool steel, the Company have been able to increase the speed of drilling to 250 revolutions per minute. Holes are sometimes drilled through twelve, fourteen, or even sixteen thicknesses of plates at one setting, and the heaviest of girders can be assembled and drilled in a few days. There are other benches with universal drills, for dealing with curved or varying forms of constructional steel work.

Hydraulic jib, or overhead, cranes command the tables, and the girders, bolted together in the case of small members, and in sections in case of large members, are taken on bogies on the 2-ft. gauge railway to the riveting department. Prior to being thus finished, they are thoroughly scraped, and oiled or painted. Another provision to obviate rust is that most of the work is carried out under cover. After the meeting surfaces have been painted or oiled, the units are riveted, the hydraulic machines for this purpose being suspended on cranes.

We have already narrated the circumstances attending the introduction of the hydraulic process of riveting by Sir William Arrol and Company, and of its superiority to handwork there is little need now to write. Its great merit is that it ensures an absolute contact at all points



Battery of Twenty-Four Radial Drills for Girder Work.

of the riveted surfaces, and the perfect filling of the holes by the stem of the rivet. Samples of hydraulic rivet work, when sawn through on the centre line of the rivet, have not shown the slightest interstice between plate and rivet. There is little limit to the application of the hydraulic riveter, and the Company manufacture a great variety of forms and sizes suitable for almost every type of

Riveting.

work. In instances where the jaws of the riveter cannot conveniently meet, the pneumatic system is applied, but experience has not shown it to be so economical. Pneumatic tools are also applied for reamering and boring in isolated cases.

The girder, or other piece of constructional work, when riveted up so far as can be done at the Works, is painted or oiled, and marked for erection purposes. It is then loaded direct into railway wagons on the sidings adjacent to the works, and despatched.

Special tools have been devised and manufactured at the works for forming plates of various shapes, such as are used now for the decking of bridges—trough, buckle, or curve plates. These are worked to shape in hydraulic presses of great power. Former dies—flat, dished, angular, or circular—are secured, one set on the underside of the

Large Press for Forming Plates of Various Shapes.

entablature or head-piece, and the other corresponding set on a table secured on the top of the ram-heads on one or more cylinders. A large tool of this type is illustrated on this page.

Near to the end of the press there is a large furnace, in which the plate is brought to a red-heat, and from which it is afterwards withdrawn by hydraulic power on to the table on the ram-heads. The plate is held in place over the dies, and by the working of the hydraulic cylinders is

forced upwards into contact with the corresponding former dies on the underside of the headpiece. As the pressure reaches 1000 tons the plate assumes the shape aimed at, and is ultimately pulled from under the press by hydraulic jiggers, being left on skids to cool and to be prepared for further operations. The press illustrated deals, at one operation, with plates 30 ft. long and 6 ft. wide, and has proved a most efficient tool. The same press is utilised for flanging work, V's being then used instead of die-stamps. There are also hydraulic joggling machines and other presses of a kindred nature for various operations.

An important department of the Company's steel work is the making of heavy caissons for the foundations of bridge piers, quay walls, etc. These form the working chambers for excavations under air-pressure. To facilitate the sinking of a caisson it has a cutting edge at the bottom; so that as the excavation proceeds and weight is added from above, the caisson, which ultimately forms the base of the pier, finds its level on the rock or firm foundation. It is ultimately filled with cement concrete. This was the procedure at the Forth Bridge, where the area of the sub-aqueous chambers under pneumatic pressure was exceptionally large, being 70 ft. in diameter. As we have already indicated in the previous Chapter, great depths have been reached in the Company's operations in the case of the foundations for bridges, and the firm have thus accumulated considerable data and acquired unique experience as to the details of design, the application of compressed-air caissons, and the plant incidental to such work.

The firm have long recognised the importance of water- and air-tight caissons, and because of this they have had very considerable success in deep foundation work. This system of sinking is not only expeditious, but brings the

greater surety which comes with the possibility of a thorough examination of the whole surface of the foundation.

The Company also manufacture the machinery connected with the pneumatic sinking of such foundations, including the air-compressors, reservoirs, locks, etc., as well as jacks and other hydraulic appliances for lowering the caisson into position. Their twenty years' varied experience

Building a Scherzer Roller Lift Span.

has resulted in many improvements in the details and design of such mechanism.

Many of the bridges built by the Company have opening spans of the swing or rotating type, of the bascule system, and of the newer Scherzer roller-lift design, and to the most representative types we have referred at some length in the preceding Chapter.

The engraving on this page shows one of the erecting yards, where a Scherzer roller-lift span is being constructed

for the Rosslare and Waterford Railway Bridge over the River Suir. The span in this case is 50 ft., but eight fixed spans make the total length of the bridge 1205 ft.

For all types of opening spans the Company construct the actuating gear, whether the motive power be steam, hydraulic or electric, and for this and other engineering

Erecting Shop in the Engineering Department.

work a separate department is arranged. There are also made here all types of hydraulic riveters and presses, cranes, hoists, etc.; special machine tools, prime movers of various kinds, coal elevators and conveyors, and the gas retort charging and discharging machinery now so extensively adopted.

The engineering department includes a machine-shop 300 ft. long and 128 ft. wide. The main bay, 50 ft. in width, is illustrated on this page. It is reserved for the

The Brass-Finishing Department.

erecting work, and the overhead traveller is of 35 tons capacity. The central bay is 30 ft. wide, and the third bay 48 ft.

It is scarcely necessary to describe in detail the various machine tools in the engineering shops. Many of them are of a special type for carrying out unusual operations. There is, for instance, a milling machine, with a special radial attachment for facing up the ends of steel-built columns and girders after the riveting is completed, so as to secure absolutely true bearing surfaces. This tool is particularly useful in connection with the construction of large rolling-lift or swing bridges. On a planing machine, again, there is an attachment for dealing with the bevelled faces of roller-paths of revolving swing bridges.

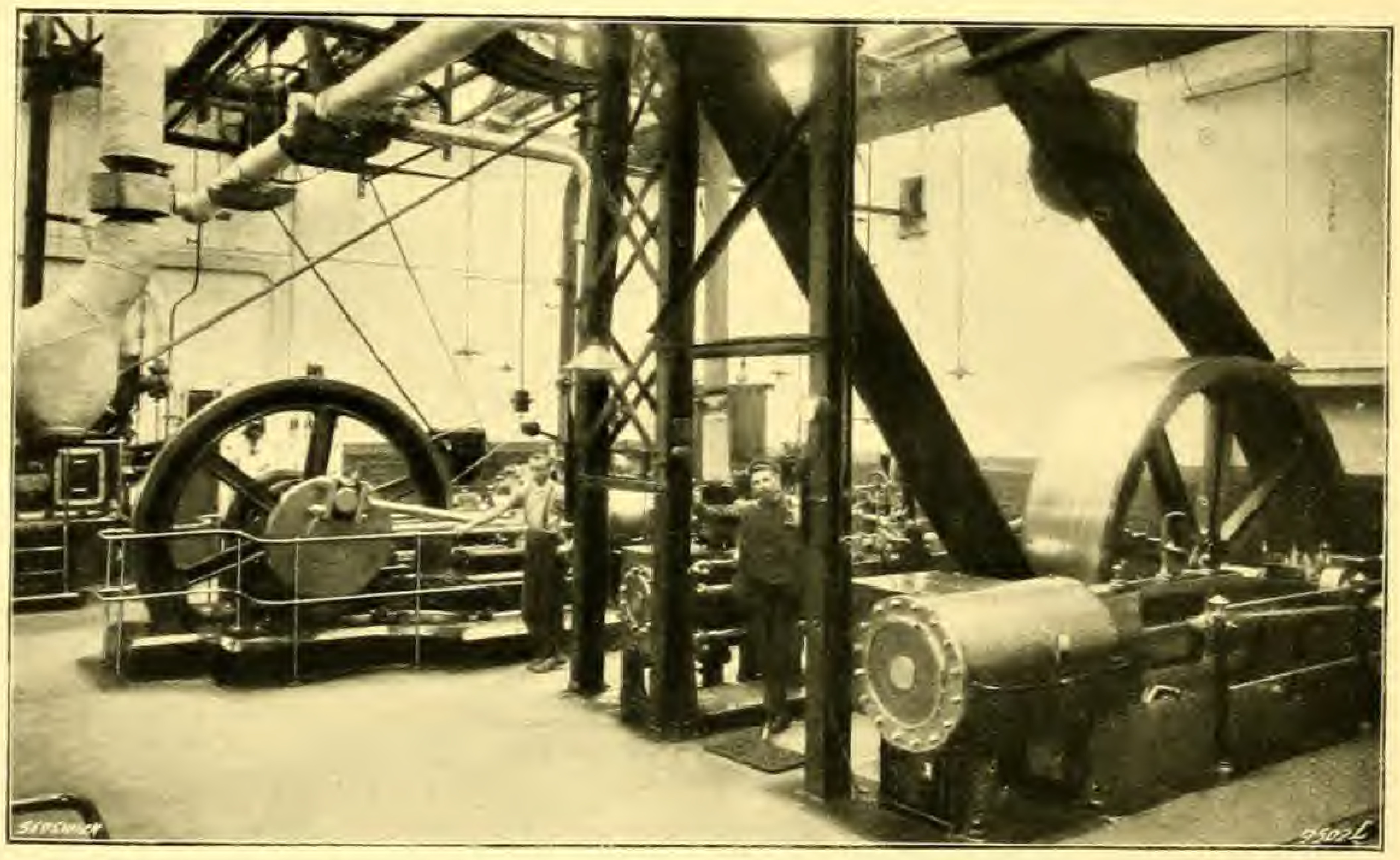
The brass-finishing department has a splendid equipment of automatic and other machine tools, and a view of it is given on the preceding page.

There is in connection with this department a brass-foundry and a well-equipped tool-room. The system of jigs and gauges is most complete, so that all parts are made interchangeable, and repeat orders can at once be met. In the smithy there are twenty-five hearths, five hammers, and three hydraulic presses.

The power plant for the whole of the works is concentrated at the Central Station, which is modern in its equipment, and has, in addition to electric generators, hydraulic pumps with accumulators, air-compressors with receivers, and other accessories. This station is illustrated on pages 51 and 53.

The main steam plant consists of a range of Babcock and Wilcox water-tube boilers, suitable for working at 200 lb. pressure, although as a rule the safety-valves are set at 160 lb. The steam is superheated to the extent of

100 deg. Fahr. The Company have installed their own system of coal-elevating and conveying plant, and with it they have made experiments and evolved improvements for the corresponding appliances which they manufacture for clients. The arrangement of the plant is shown on the plan on page 33. The coal is brought by railway trucks into the siding, and there are two hydraulic wagon-



View in Power Station.

tipping cylinders for discharging the contents into a hopper, whence the fuel falls into the boot of a bucket elevator, which passes it into another hopper. Thence it is taken by an overhead conveyor, with weighing appliances, to be discharged into the runway across the front of the boilers. From these it is finally fed into the boilers by automatic chain-grate stokers. The system thus obviates all manual labour.

There is a Corliss compound engine of 200 horse-power for driving the shafting in connection with the large tools

in line with the power station. With this exception all machines are operated by power transmitted electrically.

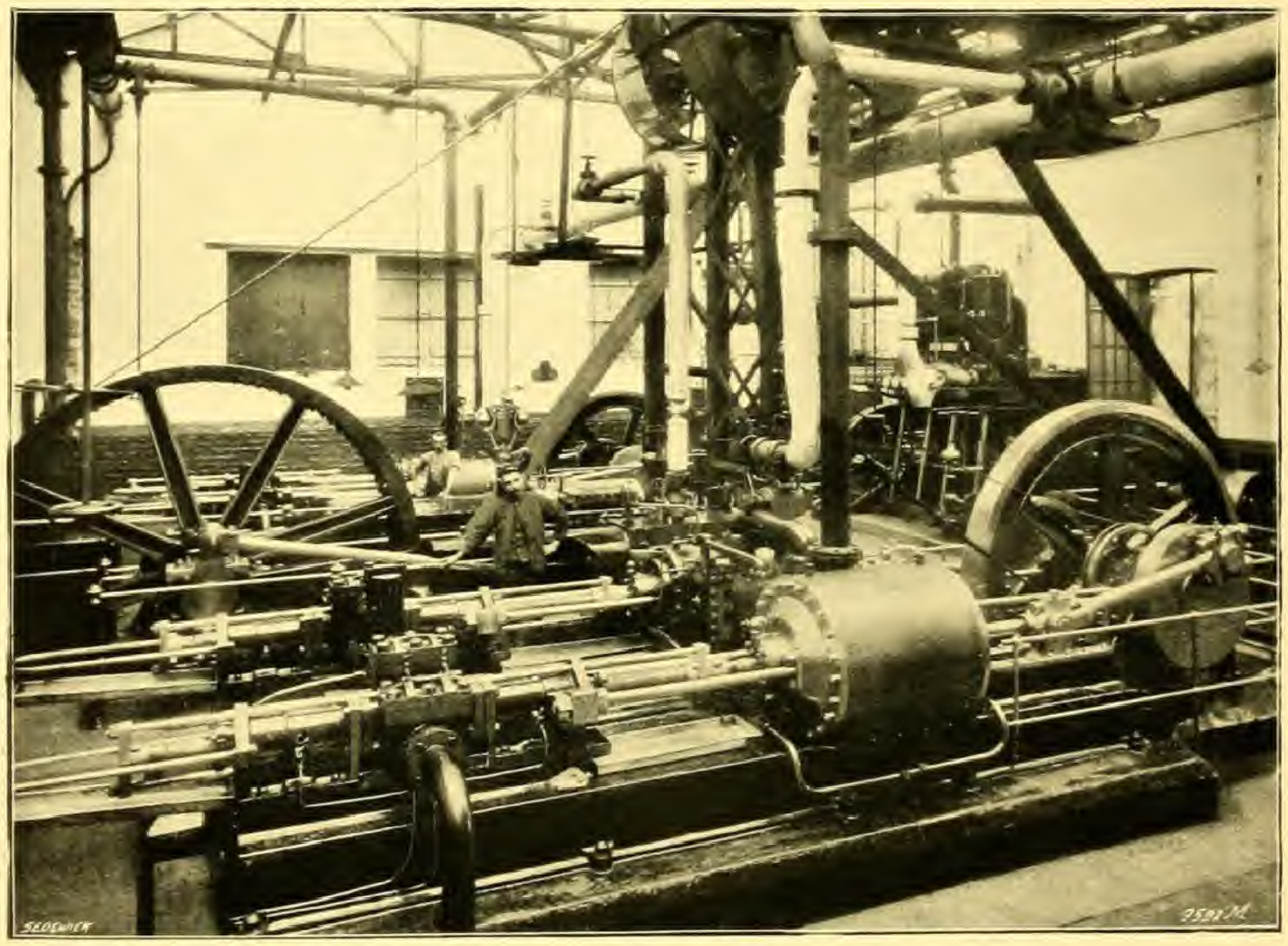
The chief electric generating set consists of a 630 indicated horse-power triple-expansion engine, coupled direct to an eight-pole compound-wound generator of 400-kilowatt capacity, which gives current for both lighting and power. There is also a 250 horse-power compound engine, coupled direct to a four-pole compound-wound generator of 150-kilowatt capacity. A third dynamo of 70-watts capacity is belt-driven from the Corliss engine. The total capacity of the generators is therefore 620 kilowatts. The electric cables from the dynamos to the main switchboard are taken along underground channels entirely separate from the steam exhaust pipes.

The current is transmitted from the main switchboard by heavy feeder cables overhead, distribution boards being situated at convenient points in the works.

A word may be said about the system of driving. The motors, almost without exception, are of the 4-pole shunt-wound type, and while many are independently connected to one machine tool, others work lines of shafting from which the tools are belt-driven. Thus, in the engineering shop, where the current is controlled from one distribution board, the principal lines of shafting are driven by four semi-enclosed 30 brake horse-power motors, running at 560 revolutions per minute. The large overhead cranes have independent motors for hoisting, travelling, and cross-traverse, the controllers in all cases being of the tramway type. In the girder-erecting department, the two batteries of radial drills, illustrated on page 31, are each run by a 50 horse-power motor, a bar-straightening machine by a motor of similar power; while in the girder-erecting shop, illustrated on page 31, there are three 10-ton and one 20-ton overhead

travellers, each having three motors ranging from 5 to 25 horse-power for hoisting, longitudinal travel, and cross-traverse respectively. The Roots blower in the smithy has a 10 brake horse-power motor.

As regards lighting, there are 150 arc lamps, and with a few exceptions these run five in series on a 250-volt



Hydraulic Engines in Power Station.

circuit. All the constructional steel departments are lighted by 10-ampere arc lamps of the open type. The drawing and tracing offices are lighted by ten inverted arc lamps, while the commercial offices have incandescent lamps of 32- and 16-candle power. In order that the offices may be illuminated while the works and power station are closed down, 137 storage cells are provided. The template and pattern shops, where there is danger of fire, are lighted

both by enclosed arcs and incandescent lamps. And here it may be said that very special precautions have been taken against outbreak of fire. One interesting feature of the electric equipment is the installation, in different parts of the works, of fifteen clocks on the Becker, Barr and Stroud system, so that workmen can always ascertain the time.

Hydraulic power is used extensively in the works. There are installed in the power station two of the Company's compound pumping engines. One of these is seen in the forefront of the illustration on page 53. The two engines differ slightly in dimensions, but they each deliver about 12,000 gallons per hour at a pressure of about 900 lbs. per square inch. The cylinders are of the tandem type. The power water is delivered into an accumulator, 18 in. in diameter by 14 ft. stroke, placed outside of the engine-room. This accumulator regulates the steam admission to the pumping-engines in the usual manner. The power water for the hydraulic apparatus is distributed throughout the works in a 4-in. cast iron main, laid 4 ft. under the ground level, so as to obviate any interference by frost. Branch pipes are laid to the various shops and tools.

From what we have written regarding machine tools it will be noted that hydraulic power is used for cranes, riveters, presses, lifts, and jacks for planing machines, etc. The return exhaust water passes to an underground tank about 30 ft. long, whence it flows to the supply tank for the engine. This latter tank, placed contiguous to the main pumps, is connected with the water from the Glasgow Corporation gravitation supply, so that from time to time the wastage in the system may be made up.

An air-compressing plant is also accommodated in the power station, consisting of a triple-expansion engine, and air-compressors capable of delivering 1000 ft. of

One of the Erecting Yards.

free air per minute at a pressure of 100 lb. per square inch. The steam cylinders are placed over the air-cylinders, as is now almost universally the case, and the air is delivered into a receiver adjacent to the compressor; thence it is distributed throughout the works in 4-in. mains, with small branches. The compressed air is used for working drilling, chipping, caulking, and riveting machines.

It will thus be seen that, from first to last, the organisation and equipment of the establishment have been developed on most efficient lines, arrived at by wide experience; and that a strictly progressive policy has been pursued, especially where carefully-conducted tests proved that a newer appliance would ensure greater accuracy and facilitate work. These two advantages carry with them the further benefit of economy.

BRIDGE BUILDING.

A Retrospect.

THE beginning of the industry of bridge building may almost be said to belong to prehistoric times. Certainly, in the work of the ancients one finds crude but suggestive applications of almost every principle embodied in the design of modern bridges, including the cantilever, the suspension, and the compression parallel girder systems. If the aboriginal was not versed in the principles of stresses and their resulting strains, he nevertheless produced useful structures within the limitations of the materials and constructional appliances available.

Wooden bridges are, of course, of greatest antiquity, and although masonry was sometimes employed, timber was used almost exclusively until the nineteenth century. Many long spans were built of timber prior to that period. Outstanding examples are the Wittingen Bridge, with a span of 390 ft., built in 1758, and destroyed by fire at the beginning of the nineteenth century ; Hanover Bridge, over the Connecticut River, in the United States, with an arch of 236 ft., built in 1796 ; and the Schaffhausen Bridge, across the Rhine, of 193-ft. span, destroyed in 1799. The longest timber span in Britain is probably the Landore Viaduct, in Wales, with a centre span of 107 ft., of which an elevation is reproduced on the next page.

Masonry was adopted more largely by the Romans, Grecians, and Egyptians ; but even in modern times the maximum span possible with stone arches is comparatively

small. The 250-ft. span of the Tezzo Bridge, over the Adda, is, perhaps, the greatest; and amongst other notable examples are the 220-ft. span of the Cabin John Bridge, Washington Aqueduct, the 200-ft. span of the Grosvenor Bridge, over the River Dee, at Chester, the 152-ft. span of

MAIN SPAN (107 FEET) OF LANDORE VIADUCT.
LARGEST TIMBER SPAN IN BRITAIN.

London Bridge, and the $141\frac{1}{2}$-ft. concrete span of the Alma Bridge, in Paris. There is much to commend masonry, and its popularity has continued for moderate spans, particularly in combination with iron or steel ribs.

The first metal bridge was constructed in 1779—a cast-iron arched span of 100 ft.—across the River Severn, near Coalbrookdale. Following in quick succession were

several others, including a cast-iron bridge over the River Wear, at Sunderland, built in 1796, with a single arch of 236-ft. span. The Southwark Bridge, with a central span of 240 ft., was built in 1819. These metal bridges proved cheaper than masonry or wooden structures; but, because cast iron was considered suitable only for compressive strains, it could not be used for girders; the stone arch was still accepted for most purposes.

First Iron Bridge ever Erected.

By the perfection of the principle of the suspension bridge it was found convenient to utilise the tensile, as well as the compressive, properties of metal. While one or two small examples date from the eighteenth century, the first notable erection was the Union Bridge, over the River Tweed, with a span of 449 ft., completed in 1820. Then followed, in 1826, the opening of Telford's beautiful structure of 570-ft. span across the Menai Straits, 102 ft. above sea level, and another at Conway of 327-ft. span. The Freiberg Bridge, over the Sarine Valley, in

Switzerland, erected in 1833-34, was of 870-ft. span, and 169 ft. above water level. The Buda-Pesth Bridge, over the Danube, of 660-ft. span, was built in 1842-49. The Clifton Bridge, over the Avon, erected in 1862-64, had a span of 702 ft., and was 250 ft. above water level.

Among the more notable modern examples of the suspension system are the Niagara Bridge of 821-ft. span; the Brooklyn Bridge, with a central span of 1595 ft.; and the new Williamsburg Bridge, also across the East River at New York, of 1600-ft. span—the largest suspension bridge completed.

The railway era brought new problems for the bridge designer. To the carrying of a dead load there were added all the difficulties of providing for a live or rolling load, associated in many cases with the necessity for providing considerable head-room beneath the bridge. The arch met the former condition, but limited the head-room; while the suspension bridge, although affording the necessary height, did not give the desired rigidity for the live load. Again the model of the ancients inspired engineers towards the introduction of parallel girders, with cross members, to take the place of the timber beams. The first of such girder bridges for railways was that completed in 1843 on the Dublin and Drogheda line—a lattice iron structure, in imitation of wooden bridges built in America. Since then we have had many variations from Robert Stephenson's rectangular tubular structure—the Britannia Bridge, with two central spans of 459 ft., and two shore spans of 230 ft. across the Menai Straits, with a clear headway of 103¾ ft. above high-water level. This was the first bridge made entirely of wrought iron. But steel of much higher ductility soon displaced the

wrought iron, and now every conceivable form of girder has been applied.

The aim of the succeeding pages is to deal with modern types, and with the introduction of various mechanisms for opening spans to pass river traffic. No apology need be made for accepting the work of Sir William Arrol and Company, Limited, as typical of modern bridge engineering, since they were not only responsible for the construction of many of the most prominent bridges, according to the designs prepared by the most famous British engineers, but have themselves in recent times done valuable work towards simplifying design and economising construction. The story of the building of some of the greatest of these structures constitutes a record of ingenuity and skill which is a credit to British engineering, and to such work of erection special reference will be made.

On the two following pages there is a list of the principal bridges built; and in the Appendix a standard specification, which the Company have prepared as the result of their extensive experience, and certain formulæ and data of interest to those engaged in the design of bridge and structural work generally.

TABLE I.—PRINCIPAL BRIDGES BUILT BY SIR WILLIAM ARROL AND COMPANY, LIMITED, GLASGOW.

Name of Bridge.	Purpose.	Type.	When Built.	Total Length.	Largest Span.	Tons of Steel or Iron Used.
				ft. in.		tons
Forth Bridge	Railway traffic over Firth of Forth... ...	Cantilever	1882-1890	8295 9½	Two of 1700 ft.	51,000
Tay Viaduct...	Railway traffic over Firth of Tay	Braced girders	1882-1887	10711 0	Eleven at 245 ft.	27,371
					ft. in.	
South Esk Viaduct...	Railway traffic over river, South Esk ...	Bowstring girders	1881	1430 0	96 0	—
North Bridge, Edinburgh	Carrying street over railway station ...	Steel arches	1895	561 0	175 0	1935
Dalmeny and Corstorphine Junction Railway bridges	Carrying Forth Bridge approach railway over river and roads	Braced and plate girders ...	1887	...	139 0	—
Reconstruction of Burnbank Tunnel Bridge	Carrying N. B. Railway over road	Plate girders	1904	56 0	56 0	166
Bridge over road at Camelon	Do. do.	Do.	1901	103 6	52 6	143
Kidston Mill Bridge	Do. do.	Plate bowstring girders ...	1894	72 0	65 0	47
Renewal of Hexham Viaduct	Do. do.	Plate girders	1894	90 0	48 0	66
Renewal of bridge over River Lyne ...	Carrying N. B. Railway over River Lyne ...	Do.	1896	136 0	45 0	130
Original Clyde Viaduct	Caledonian Railway over Glasgow Harbour	Braced girders	1878	700 0	200 0	3,000
New Clyde Bridge	Do. do.	Do.	1905	752 0	200 0	11,000
Ann Street Bridge	Caledonian Railway over street	Plate girders	1901	46 0	40 0	300
Milnwood	Carrying road across Caledonian Railway...	Do.	1900	82 0	74 6	125
Hamiltonhill extension (twelve bridges) ...	Carrying roads across Caledonian Railway, etc.	Do.	...	...	76 0	1265
Aberdeen widening bridges, Caledonian Railway	Carrying railway over streets	Do.	1904	...	88 0	550
King Street Bridge, Glasgow	Caledonian Railway over street	Do.	1906	68 9	...	630
Nelson Street Bridge, Glasgow	Do. do.	Do.	1906	68 0	...	600
Wallace Street Bridge, Glasgow	Do. do.	Do.	1906	68 0	...	510
Bridge over the Clyde, East Clyde Street and Adelphi Street, G. & S. W. Railway	Railway over river and two streets... ...	Main spans, steel-arched girders; side spans, braced girders	1899	557 0	84 6	2405
Bridge over Dunlop Street...	Carrying G. & S. W. Railway across streets	Plate girders and braced girders	1899	45 0	...	180
Bridge over Stockwell Street	Do. do.	Do. do.	1899	76 6	65 6	330
Bridge over Abbotsford Place	Do. do.	Do. do.	1899	84 6	...	
Bridge over Salisbury Street	Do. do.	Do. do.	1899	71 6	...	
Bridge over Cumberland and Surrey Streets	Do. do.	Do. do.	1899	152 0	...	
Bridge over Main Street	Do. do.	Do. do.	1899	116 0	...	
Bridge over Greenside Street	Do. do.	Do. do.	1899	46 0	...	2615
Bridge over Rutherglen Road	Do. do.	Do. do.	1899	66 6	...	
Bridge over Govan Street	Do. do.	Do. do.	1899	67 0	...	
Bridge over Eglinton Street	Do. do.	Do. do.	1899	76 0	...	
Bridge over Salkeld Street, Caledonian Railway	Do. do.	Do. do.	1899	105 0	52 0	
Cairn Valley Light Railway bridges ...	Carrying roads, and railway over roads ...	Plate girders	1902	...	88 0	265
Great Western Road Bridge, Glasgow ...	Carrying roadway over River Kelvin ...	Cast-iron arches...	1890	295 0	91 0	—
Glasgow, Bothwell, Hamilton, and Coatbridge Railway bridges	Carrying roads over Caledonian Railway and railway over roads	Plate and braced girders ...	1875	...	95 0	—
Girvan Bridge	Carrying road over river	Plate girders	1893	176 0	76 9	107

Spring Creek bridges	Carrying road over river	Braced girders	1879	300 0	100 0	—
Manchester Ship Canal bridges (twelve bridges)	Carrying railways across ship canal, etc. ...	Do.	1890	...	299 0	—
Dalguross Bridge, Comrie	Road bridge over River Erne	Cantilever	1904	200 0	90 0	182
Swale Bridge	Railway and roadway over channel ...	Scherzer rolling-lift bridge ...	1905	122 0	72 0	962
Suakin Berber Railway bridges	Carrying Sudan Railway	Braced girders	...	6495 0	105 0	4725
Barrow Viaduct	Carrying railway across River Barrow ...	Do.	1905	2139 0	148 0	5385
Suir Viaduct...	Carrying railway across River Suir... ...	Do.	1906	1209 0	148 0	2320
Rosslare Harbour Viaduct...	Carrying railway out to pier	Plate girders	1905	910 0	82 6	1200
Bridges over River Nile at Cairo	Carrying road across River Nile from Cairo to Ghizeh	Braced and plate cantilevers with central girders	1906	2254 0	140 0	8300
Aberdeen Harbour Swing Bridge... ...	Carrying roadway and rail tracks over dock entrance	Braced bowstring girders ...	1904	152 0	...	571
Bonar Bridge	Carrying road over Kyle of Sutherland ...	Do. do. ...	1894	333 0	151 0	360
Redheugh Bridge	Carrying road over River Tyne	Braced girders	1900	1190 0	248 0	2750
Shoreham Viaduct	Carrying railway over river	Plate girders	1892	439 0	79 0	1940
Swing Bridge at Barnstaple	Carrying road over River Yeo	Do.	1899	60 0	37 6	50
Bridge at Caputh Ferry	Carrying road over River Tay	Braced girders	1889	419 0	140 0	—
Tower Bridge, London	Carrying road over River Thames	Suspension and double bascule	1886-1894	880 0	270 0	14,000
Keltny Burn Bridge, Aberfeldy	Carrying road over Keltny Burn	Masonry approaches and steel arch	1896	148 0	60 0	40
Mary River Bridge	Carrying railway over river...	Braced girders	1879	450 0	150 0	—
Kirkcaldy and Dysart Water Works, Cow Bridge	Carrying road and piping across stream ...	Plate girders	1896	70 0	70 0	40
Millhaugh Bridge, River Almond	Carrying road over River Almond	Braced girders	1894	69 0	69 0	39
Shireoaks and Laughton Railway bridges	Carrying railway and roads...	Plate girders and troughs ...	1902	...	48 0	263
Swing Bridge at Bowling	Carrying railway over canal...	Braced girders	1897	104 0	74 3	367
Swing Bridge at Kilbowie	Do. do.	Do.	1901	152 7	116 0	415
Swing Bridge at Lambhill	Do. do.	Plate girders	1901	85 0	56 0	31
Keir Bridge	Carrying road across river	Braced bowstring	1897	110 0	110 0	26
E. & W. India Docks Swing Bridge ...	Carrying railway over dock entrance ...	Braced girders	1892	121 0	78 0	185
Victoria Docks Swing Bridge	Foot-bridge over dock entrance	Do.	1894	220 0	144 0	115
Bridge over River Annan at Hoddom Castle	Foot-bridge over river	Lattice girders	...	228 0	46 0	—
No. 1	Carrying N. E. Railway on approaches to bridge over River Wear, over streets, etc.	Plate girders	1906	22 0	...	9000
No. 2	Do. do.	Do.	1906	30 0	...	
No. 3	Do. do.	Do.	1906	48 0	...	
No. 4	Do. do.	Do.	1906	69 0	...	
No. 5	Do. do.	Do.	1906	85 0	...	
No. 6	Do. do.	Do.	1906	58 0	...	
No. 7	Do. do.	Braced girders	1906	239 0	...	
No. 8	Do. do.	Plate girders	1906	25 0	...	
No. 9	Do. do.	Do.	1906	23 0	...	
Bridge over River Wear	Carrying N. E. Railway and roadway over river	Braced girders	1906	1560 0	330 0	
Walney Bridge	Carrying road and tramway over channel...	Braced girders with rolling-lift span in two leaves in middle	1906	1123 0	145 0	2700
Blackfriars Bridge Widening, London ...	Carrying roadway and tram lines over river	Steel arches on masonry piers...	1907	1000 0	185 0	3000

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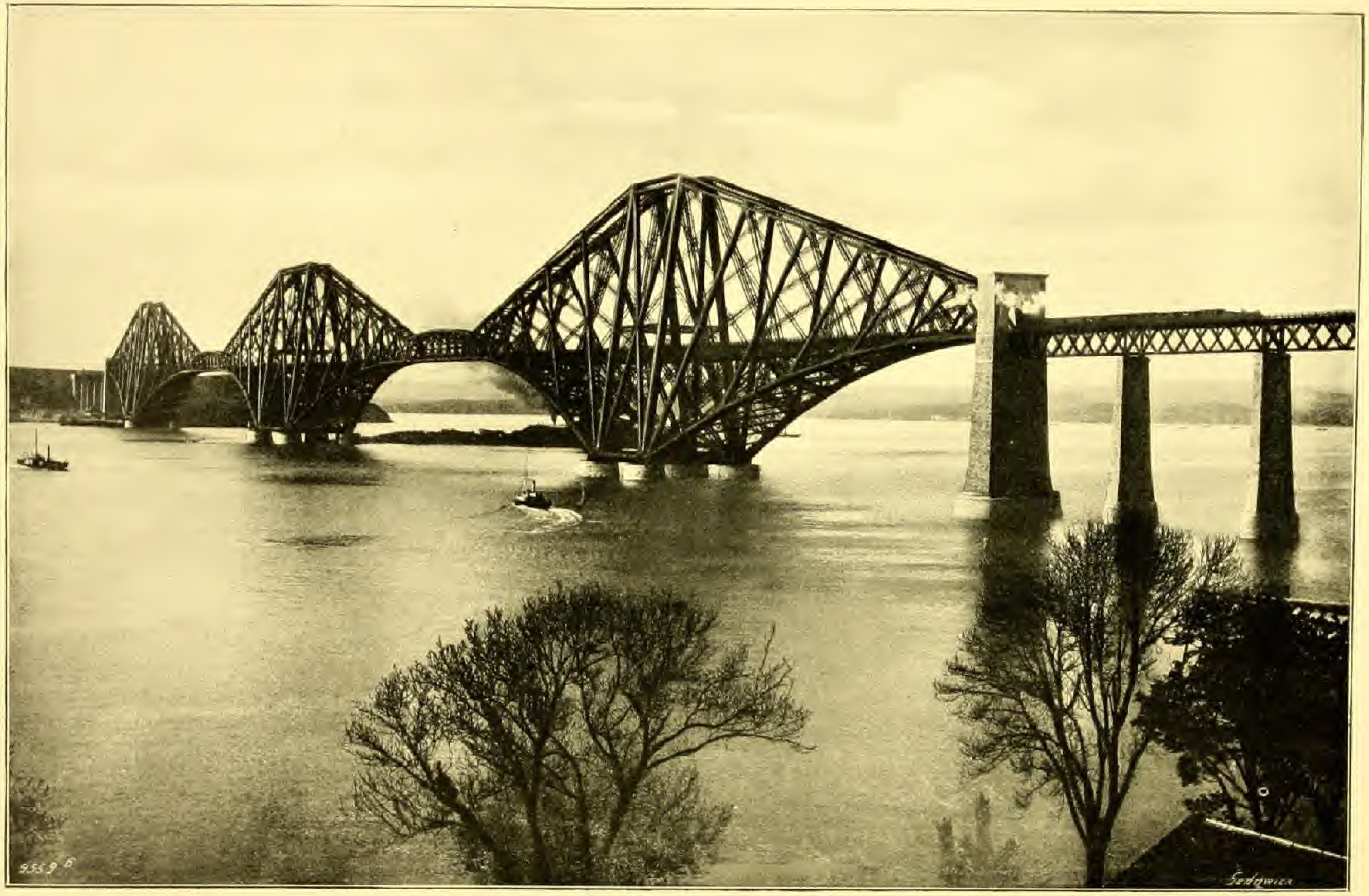
The Forth Bridge.

THE Forth Bridge,[1] probably the most famous, and certainly the largest, of all railway structures, is the most notable exemplification of the cantilever principle.

The idea of bridging the Firth of Forth was suggested nearly two centuries ago. Early in the last century the boring of tunnels under the estuary was proposed. But it was not until 1873 that a definite scheme was submitted to Parliament, with the view of connecting Edinburgh with the rich mineral and agricultural area north of the Firth. The proposal was favoured, because it obviated a long détour westward, *viâ* the Alloa and Stirling bridges across the river. It is true that there was a train-transporting ferry across the Firth at Burntisland, but this communication was intermittent. The 1873 scheme was for a suspension bridge, but for various reasons it was never carried out.

The present structure owes its conception to the genius of the late Sir John Fowler, Bart., and of Sir Benjamin Baker, K.C.B., K.C.M.G., F.R.S.; and as soon as Parliament, in 1882, authorised the carrying out of the design, a contract was signed, and the work of construction, under Sir William Arrol's direction, was commenced in December of the same year. The desiderata in design and construction were (1) rigidity, vertically under a moving load, and laterally under wind

[1] ENGINEERING, vol. xlix., page 213.



The Forth Bridge.

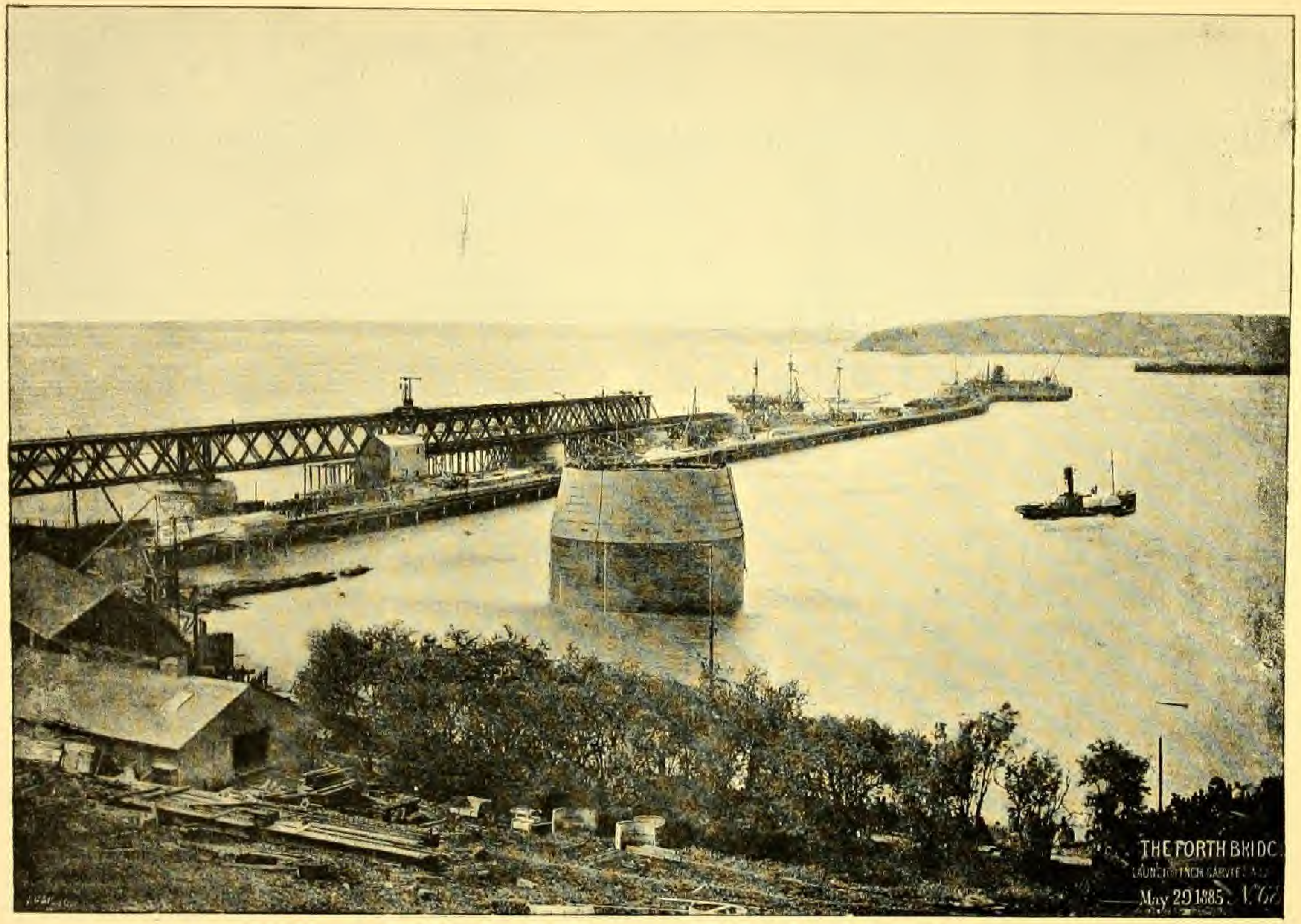
pressure; (2) facility and security in construction to secure immunity from accident at any stage of erection; (3) the use of material proved by long experience; and (4) economy consistent with the fulfilment of the preceding conditions.

The water-way at the site of the bridge is 5700 ft. wide, but the total length of the bridge is 8295 ft. 9½ in., as approaches on a rising gradient had to be constructed to ensure a height of quite 150 ft. above high-water level in the centre, for the passage of steamers and sailing ships.

The main feature of the structure is the three double cantilevers, with two intervening centre girders. The total length covered by the cantilever structures, with their connecting girders and piers, is 5349 ft. 6 in. The South approach viaduct, with ten spans and four arches, accounts, with abutments, for 1978 ft.; and the North approach viaduct, with five spans and three arches, similarly makes up 968 ft. 3½ in.

The site was selected partly because the island rock of Inchgarvie formed a natural base for the construction of the central pier of the bridge, as shown in the engraving on page 67. The two other piers, on the Fife and Queensferry shores respectively, necessitated extensive work in sinking into the bed of the river caissons for the foundations.

Resting upon independent piers, formed of iron, concrete, and masonry, great vertical steel columns, four for each cantilever, rise to a height of 361 ft. above high-water level, and carry the 12-ft. tubes forming the main members of the cantilever brackets, with their vertical and diagonal bracing. The piers were sunk into the bed of the river to a depth, in some cases, of 50 ft. and 60 ft.



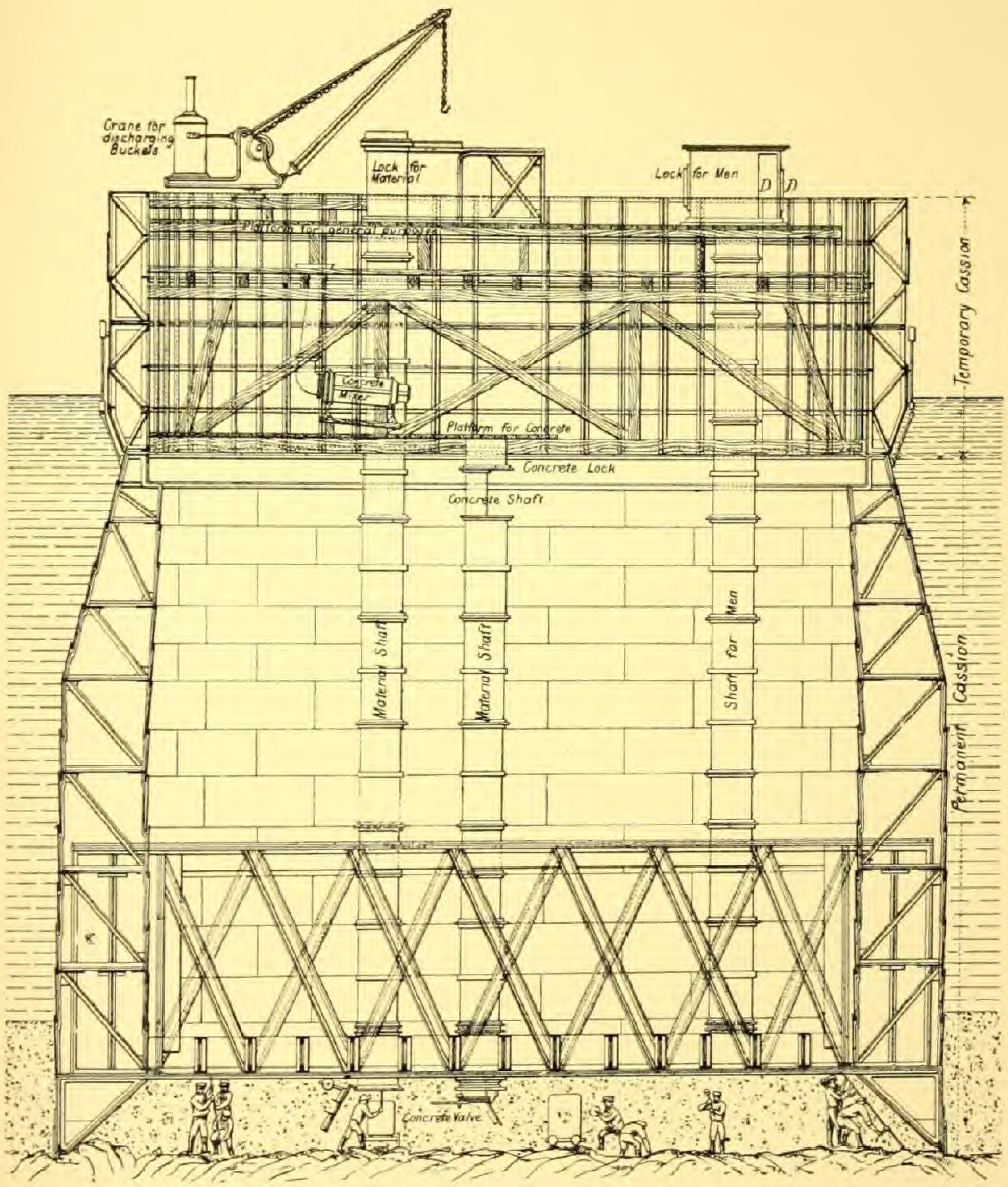
Launching of Caisson for the South-West Pier of the Forth Bridge.

to ensure a firm foundation, and were of sufficient area to limit the pressure to about $5\frac{1}{2}$ tons per square foot under the dead and live loads, and a wind pressure of 56 lb. per square foot.

The caissons, which at their base have a diameter of 70 ft., were constructed on shore, of plates, stiffening angles, and beams; and, when of sufficient height, were launched into the Firth; and, weighing in some cases 500 tons and drawing $10\frac{1}{2}$ ft. of water, were towed to the point where they were to be sunk. A platform was formed on top to carry air-locks, cranes, workshops, etc. When these were arranged, the process of sinking the caissons through the muddy silt or clay was proceeded with.

The construction of the caisson—probably the largest so far sunk under compressed air—is well shown on the opposite page. When the caisson had been sunk into the bed of the estuary, air pressure was introduced into the working chamber at the base, and the water and silt within it were ejected through pipes with outlets above high-water level. Workmen were then able to enter the chamber for the excavation of the heavier material, so that the caisson could sink into its permanent position. When the hard boulder clay was reached, it was found to be so dense and tenacious that only slow progress could be made by pick and spade. Sir William Arrol therefore devised a tool which, while it could be moved by hand, was nevertheless operated by power; and as it is one of many similar instances of invention, this tool may be briefly described.

It was an hydraulic spade, having a ram, to which a cutting and lifting surface was attached. The ram fitted into the hydraulic cylinder which represented the



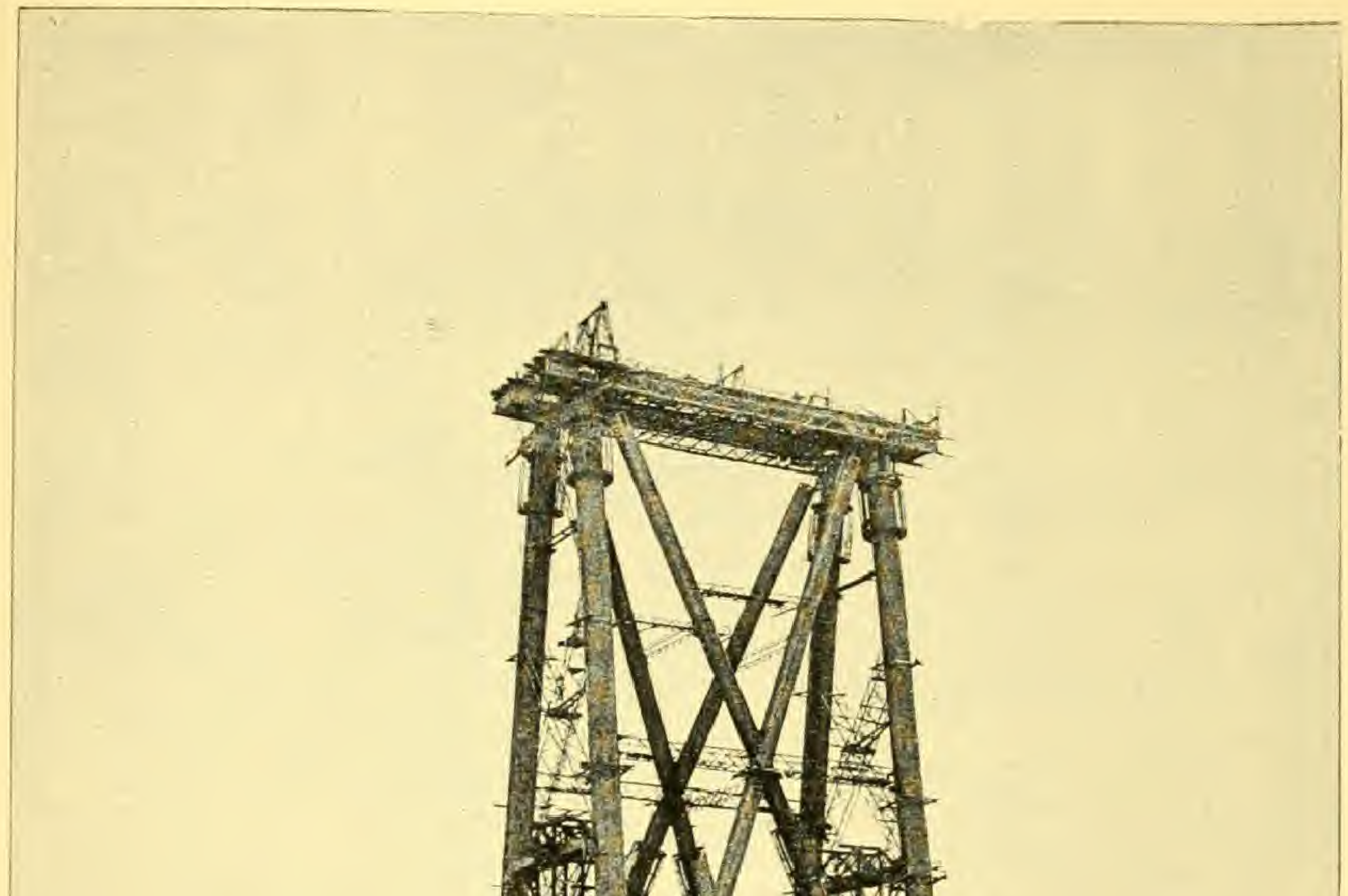
Section of Caisson with Air Locks and Working Chamber.

handle of the spade. On the cylinder top there was a headpiece, which was set against the roof of the working chamber, and with this mechanism it was easy to deal with the hardest clay. From 6000 to 7000 cubic yards of material were excavated, the caissons being sunk for about 20 ft. into tenacious material.

The time occupied in the sinking of each caisson averaged about three months, and in this period the immense cylinder was sunk to from 70 ft. to 90 ft. below high-water level. Two weeks sufficed for the filling in of the working chamber with concrete, where such was adopted. The upper part of each pier was built of masonry with rubble filling, temporary caissons being erected to exclude the water. And thus, practically hidden away from any view of the structure, there are, in the twelve piers constituting the supports of the steel superstructure, over 500 tons of steel, 2600 tons of iron, 44,000 cubic yards of cement concrete, and over 176,000 cubic feet of granite, etc.

The time taken for the completion of each pier, which represented a weight of about 21,000 tons, averaged about fifteen months. The superstructure was of coursed granite, with hearting of Arbroath rubble.

On the four piers were erected steel tubular uprights, 343 ft. high, to carry, from their base the main tubular members of each great cantilever arm, and from the top the braced upper members and diagonal ties. Tubes were preferred, because experience showed that this section gave greater resistance to compression stresses, weight for weight, than other forms. The steel columns are vertical only in elevation; in cross section they have an inclination of about 1 in 7½, being 120 ft. apart at the bottom, and 33 ft. at the top. This batter is maintained throughout the



Fife Pier Erected to Full Height.

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whole length of the bridge, as shown in the engraving on page 77. The four columns, each rising from a separate masonry pier, are thoroughly braced laterally and diagonally, forming a tower of immense strength and weight—7036 tons in the case of that built on Inchgarvie, and 4815 tons in each of the others—well able to resist the enormous stresses resulting from the combined influences of dead load, live load, and wind pressure upon the tower itself and upon the cantilevers projecting from it.

All these stresses are concentrated on the circular masonry piers, immediately over which are the main junctions or skewbacks. These are the gathering-points of five tubular and five lattice-girders of immense strength. Each column terminates at the foot in a flat plate forming the main bed-plate, which rests upon or slides along another, a lower, bed-plate, 37 ft. long and 17 ft. wide, fixed on the masonry pier. Only one skewback out of the four comprised in each tower is fixed; the other three, being free to slide, yield to the influence of temperature and of lateral deflections produced in the cantilevers by wind pressure. Provision is also made at four points in the length of the bridge, at rail level, for temperature and deflection movements.

The work of building the cantilever arms commenced simultaneously from the top and bottom of each of the four uprights of each tower. The bottom member is circular in cross section, each portion from junction to junction being a straight line, and passing into the next portion at an angle. These tubes decrease in diameter from 12 ft. at the skewback to 6½ ft. at the end of the fourth panel, after which they gradually become rectangular in form. The top member, being always in tension, is straight. Between the two there are diagonal struts and

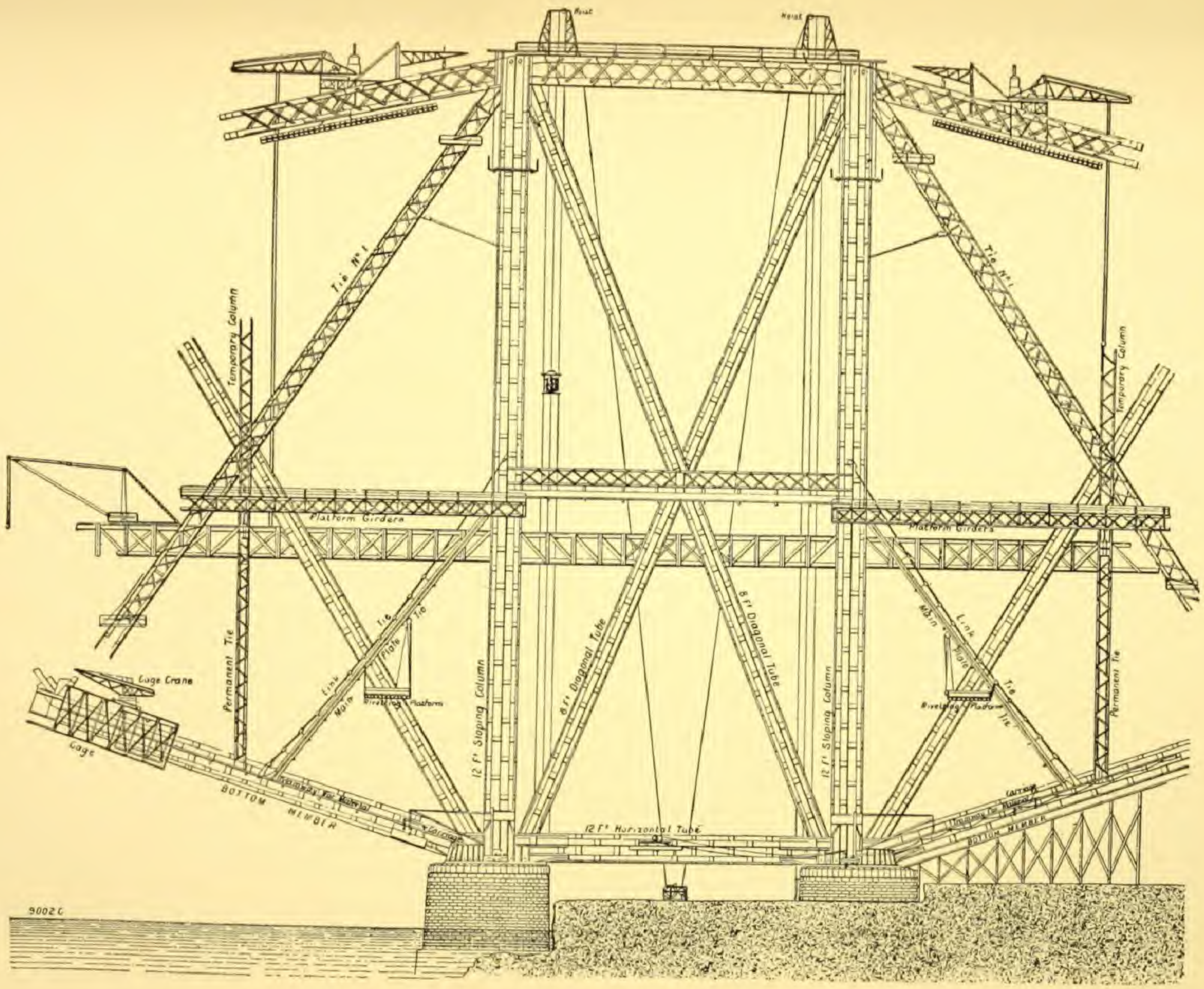


Diagram Illustrating the Building of one of the Piers and Cantilevers.

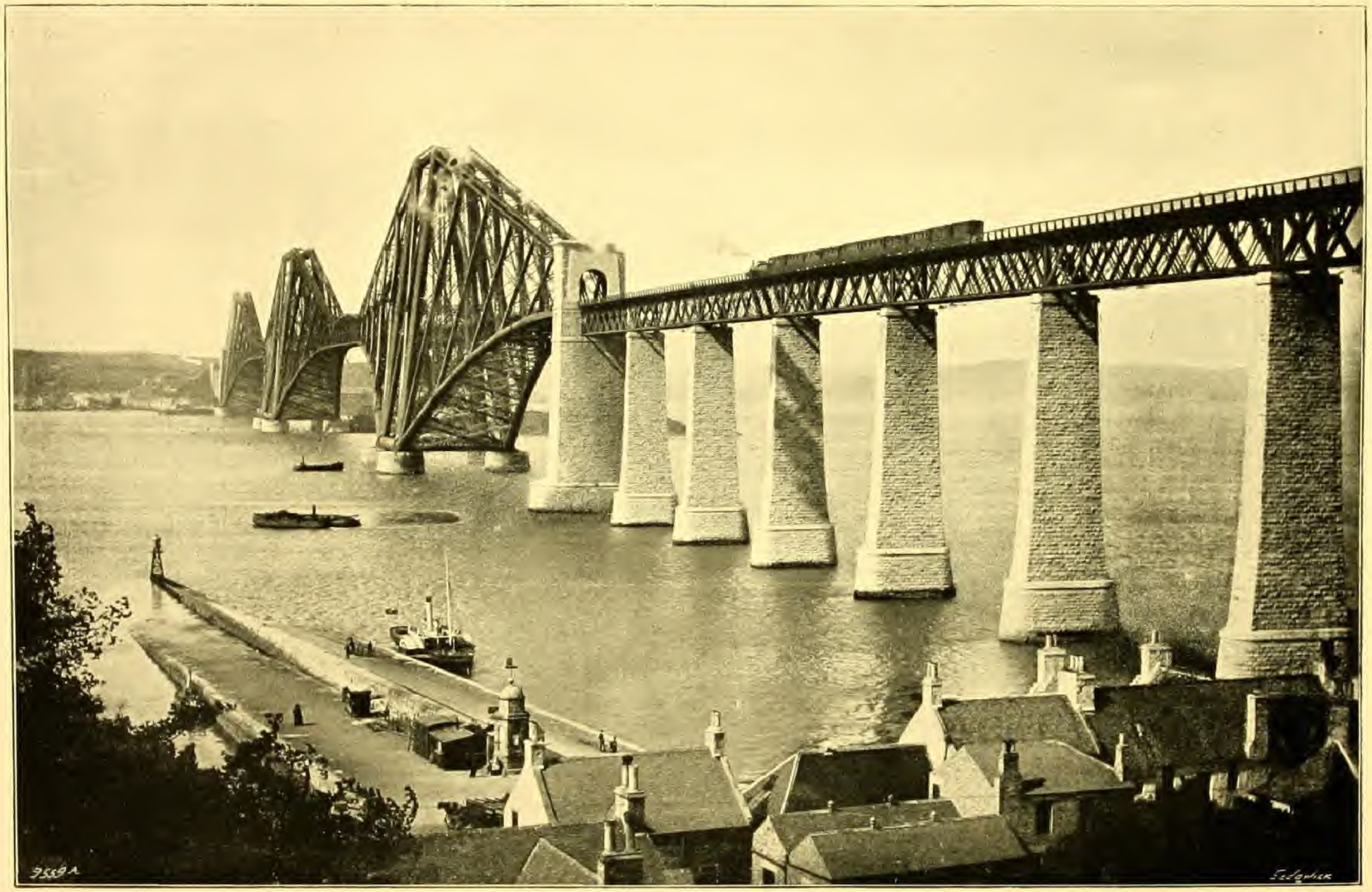
ties, while in the cross section there are corresponding diagonals between the struts.

The internal viaduct, which carries the permanent way for a double line of rails and a footpath on each side, consists, in the main, of two lattice-girders set at 16 ft. centres, and of depths varying according to the length of the span. This internal viaduct is carried by transverse plate girders between the vertical columns of the piers, and by vertical supports extending upwards from the point of intersection of the vertical wind-bracing, between the main booms of the cantilever brackets.

The erection of the superstructure, involving the use of 51,000 tons of steel, within three years was a remarkable performance. Two thousand tons of steel were worked into the structure during some of the months.

Such a performance is proof of the foresight of the contractors, and the efficiency of their management and of the plant they devised. It would be impossible, in the amount of space we are able to devote to this one bridge, to describe any of the tools invented by the firm to achieve their object. The immense tubes of the structure required special drilling machines and plate-edge planers; while in the work of erection the problems associated with the lifting platforms, the securing of stages, the suspension of hydraulic tube-riveting machines and other tools, were scarcely less important than those associated with the design of the structure. The work of riveting was itself an important matter, and for closing the 6½ million rivets used, a great variety of machines had to be introduced to suit difficult positions.

A few words may be said regarding the approach viaducts. That from the south is well illustrated on the opposite page. In this case there are ten spans of 168 ft.



The Southern Approach Viaduct.

each, four arches of 66 ft. each, and abutments 34 ft. wide; while on the north approach viaduct there are five spans of 168 ft., three arches of various sizes, with abutments 14 ft. $3\frac{1}{2}$ in. wide. The immense granite piers carrying the girders to form the viaducts have each a base 61 ft. by 31 ft., and are built solid, forming very handsome structures, suggestive of great strength. The abutment has a base of 108 ft. by 57 ft. Between the anchorages for the shore cantilevers there is, for the passage of trains, an archway, 24 ft. wide.

In the bridge, with approaches, there were used 51,000 tons of steel work, 65,000 cubic yards of cement concrete, 49,000 cubic yards of rubble, and 750,000 cubic feet of granite.

The bridge, which was completed in the early days of 1890, and opened by his Majesty the King, then Prince of Wales, on March 4, 1890, occupied in its construction seven years, and the cost worked out at £3,000,000. Over it there pass per day, on an average, 221 trains, 161 being passenger and 60 goods trains; and the joint owners—the North British, the Midland, the North-Eastern, and the Great Northern Railway Companies—have every reason to be proud of their enterprise.

The Tay Bridge.

THE idea of bridging the Firth of Tay first took form in 1849, but it was not until 1870 that a Bill authorising the project was passed by Parliament, and the first bridge, 10,700 ft. long, was opened for traffic in 1878. On the 28th of December of the following year, the thirteen central spans, each of 245 ft. and with a clear height above high-water level of 88 ft., collapsed during a violent gale.

The utility of such a bridge, however, had been clearly demonstrated, and a new engineer, Mr. W. H. Barlow, and new constructors, Sir William Arrol and Company, Limited, were chosen for the new bridge. Its construction was authorised by Parliament in 1880, was commenced in June, 1882, and completed in 1887. It is 10,711 ft. long, with a width on top of 24 ft., to accommodate two lines of railway.[1]

The features which excited most interest in the new structure were the foundations and the wind-bracing, and severe gales have proved their efficiency. Public confidence, shaken by the collapse of the first bridge, has long since been restored. This is proved by the fact that whereas in 1889 the greatest number of trains crossing in a day was 104, the average is now 163, of which 119 are passenger and 44 goods trains.

[1] See ENGINEERING, vol. xxxii., page 575; vol. xxxix., page 689; vol. xlii., page 664; "The New Tay Bridge," by Crawford Barlow, B.A., M. Inst. C.E.

Prior to the opening of the bridge, the North British Railway Company paid each year to the Caledonian Company, for conveying through traffic over their line *viâ* Perth, a sum equal to 5 per cent. on the cost of the new bridge.

The southern approach consists of three brick arches, with a fourth of exceptional dimensions to form the main abutment. For the northern approach there are eight spans, six on the foreshore between high and low-water marks, and two on land. Three are arches, and the others are built up of girders, resting on cast-iron columns, with granite and brickwork pedestals.

The bridge across the Firth has 74 spans: 13 over the central channel, with 24 to the north, and 37 to the south, shores. The permanent way is carried on the top of the girders in the case of the northern and southern spans, which vary from 56 ft. to 118 ft., but it is on the bottom booms in the case of the central spans. This ensures a clear headway of 79 ft. for the passage of ships. Eleven of the central spans are 245 ft., and two are 227 ft.

In the evolving of appliances and plant for the execution of the work great ingenuity was exercised in this, as in other large undertakings carried out by Sir William Arrol and Company. This care and foresight contributed largely to the economy and rapidity of construction, as well as to the efficiency of the completed structure. The story of the building of the bridge is, therefore, of interest.

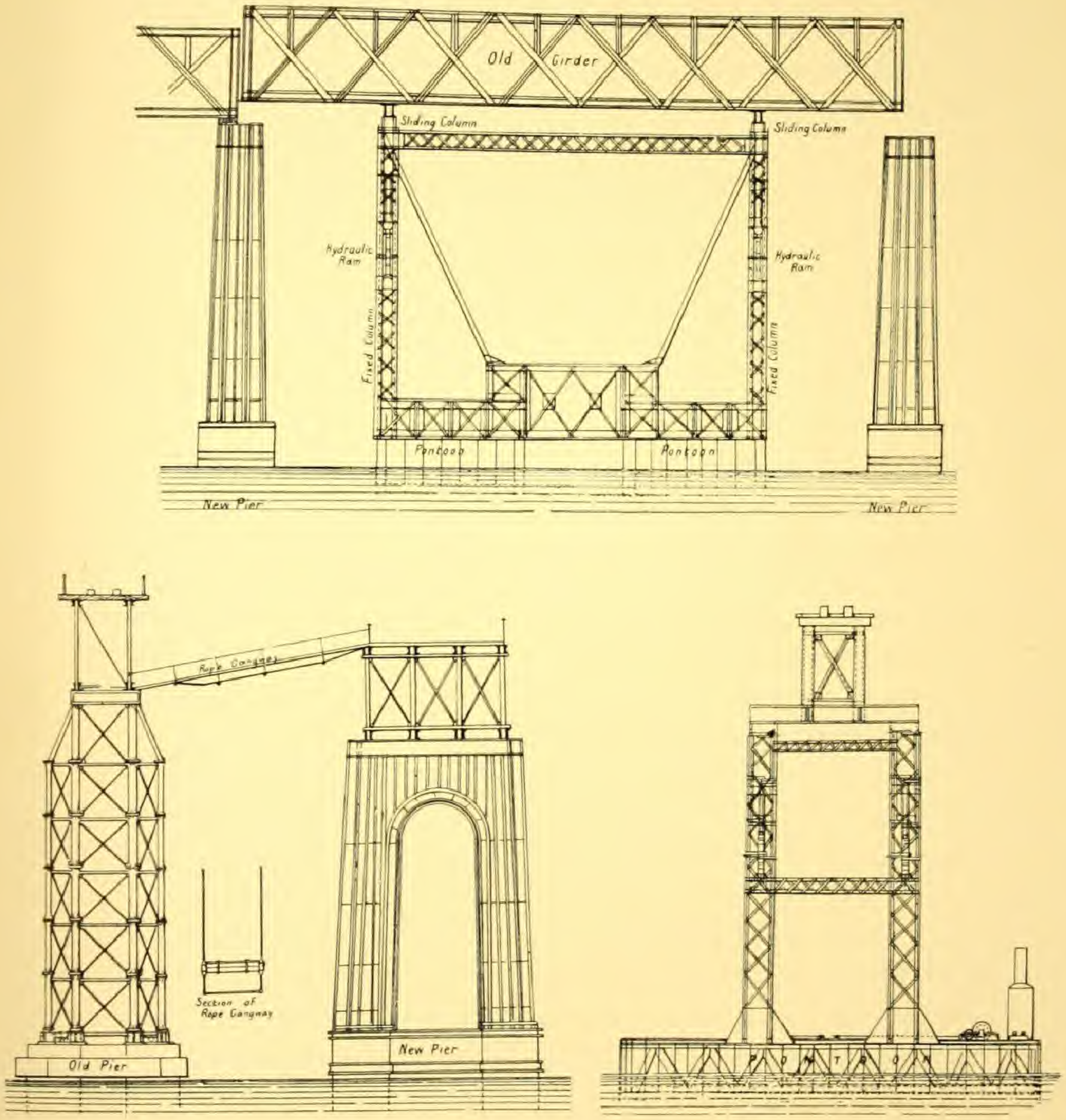
The piers, which cost £4000 each, are founded 20 ft. below the bed of the Firth, to provide, amongst other things, against scouring action. The base is proportioned to limit the load to 3½ tons per square foot; the test—a dead—load was exactly double this. Each pier consists of two iron cylinders, from 26 ft. to 32 ft. apart, joined together 1 ft. 6 in. above high-water level by a horizontal cast-iron girder 8 ft.

The Tay Bridge.

deep, or, as in the larger piers, by smaller cast-iron girders, covered with masonry and brickwork. These transverse girders have a width corresponding to the diameter of the cylinders. This arrangement increases resistance to the action of floods and floating masses. From this level the piers are octagonal shafts built up of wrought-iron plates, angles, and tee-bars, with internal diaphragm plates at short intervals in the height. These shafts are united at the top by the semicircular arch on which the main girders of the bridge rest. The arches distribute equally the dead and live loads and lateral stresses, which are transmitted through the cylinders to the foundations.

The sinking of the foundations was carried out with expedition and security. On the site of each pier there was formed, as described on page 10, a working platform on a pontoon, so arranged as to leave vacant the water area through which the two cylinders were to be sunk. Successive lengths of each cylinder were constructed on shore, and conveyed in barges to the pontoon, on to which they were lifted by cranes to be subsequently lowered into position by hydraulic gear. As soon as a cylinder touched the bottom, grab dredges were used to excavate the material in the interior. Where necessary it was loosened by hydraulic jets, acting at the cutting edge, and manipulated by divers. The cylinders were forced downwards, partly by superimposed weights, and partly by hydraulic jacks mounted on the pontoon. The pontoon was kept rigid by its connections with anchoring legs which had been driven into the bed of the Firth. In these there were holes for the insertion of pins, to secure the pontoon at a level coincident with the height of the cylinder work.

When the cylinders had been sunk to the required



Transferring Girders from Old to New Piers.

depth they were filled with concrete, packed, where necessary, by divers. The girders connecting the two cylinders of each pier just above high-water level are enclosed in brickwork. Above this level are the octagonal shafts, with a connecting arch transverse to the line of the bridge at the top. The wrought iron plates, angles, tee-bars, and diaphragm plates forming these shafts were made to template at the Glasgow Works of the Company, and brought to the site ready for erection and riveting. The piers are illustrated on the preceding page.

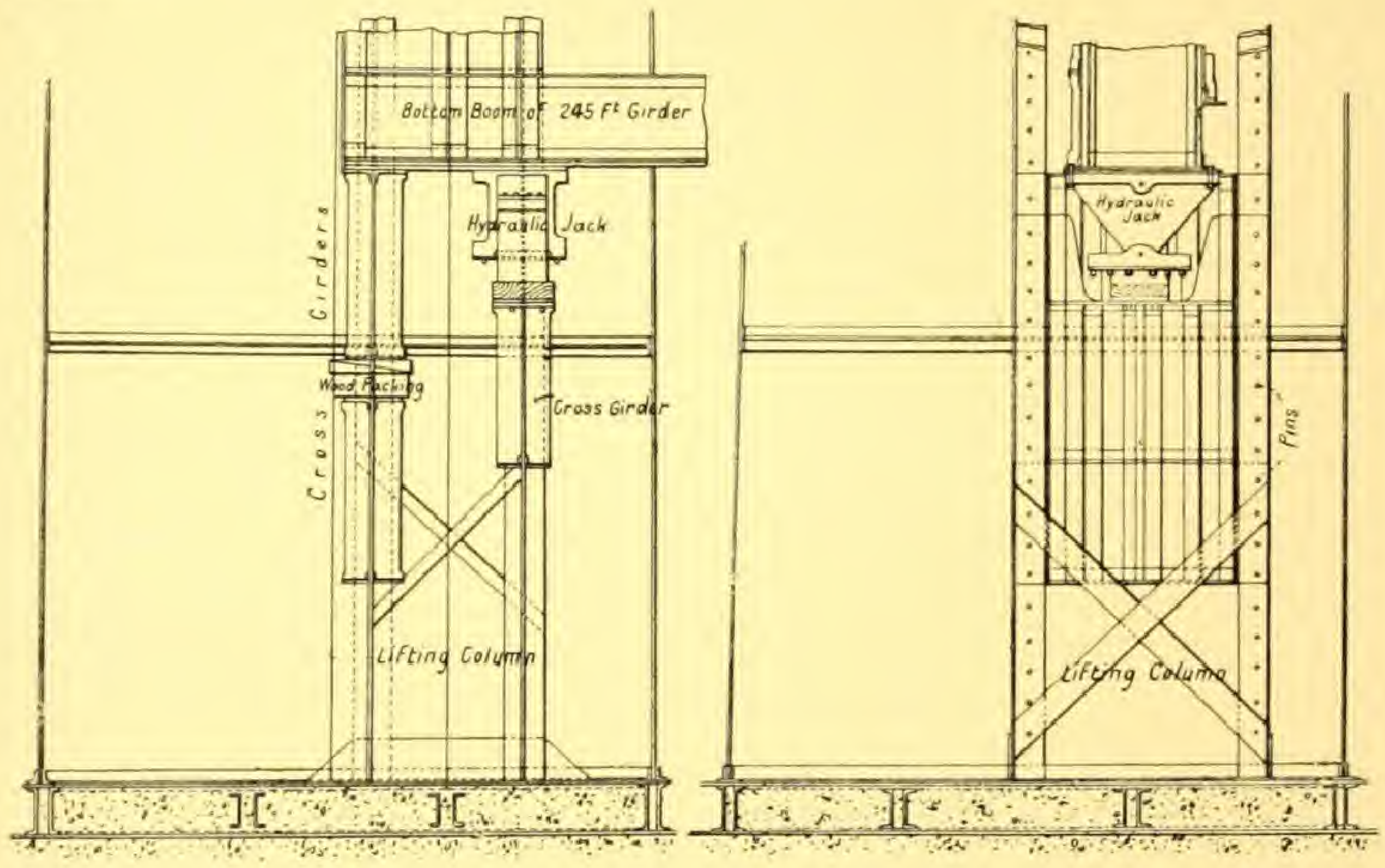
It was decided to use many of the girders of the old bridge, which was 60 ft. distant from the new structure. For transferring these girders from the old to the new piers, pontoons were constructed with a platform on telescopic supports, so that the height could be varied to suit the level of the bearings for the girders. The pontoon was moored under an old span at low water, and, floating on the rising tide, it lifted the span, which was then conveyed between the new piers, and lowered by hydraulic jacks into its new position.

The new bridge has two railway tracks, the old bridge had only one. Four parallel lines of girders were therefore provided instead of two. The old girders were used as the outer members. The two inner lines of girders are new, and these were conveyed to their position by a traverser running on rails laid on the top of the outside—the old—girders, and were subsequently lowered into position by hydraulic gear mounted on the traverser.

The thirteen central spans are entirely new, and for these and all other work the metal was so arranged that the strains would not exceed 4 tons per square inch. These immense spans of heavy lattice construction weighed 514 tons each, and cost £47 per lineal foot. They were

built complete on pontoons, which fitted compactly into docks in a wharf on the shore of the Firth.

So perfect was the constructional plant that seventy-two days sufficed for the erection and riveting of each span, four hours for the floating of it into position, and twenty-one days for the hoisting of it on to its bed-plates.



Raising Centre Span Girders on to New Piers by Hydraulic Jacks.

The span was so erected on the pontoon that when it was moored between the piers, and the tide fell, the bottom of the girder work was level with and rested upon the top of the brickwork surrounding the girders between the two cylinders of each pier, 1 ft. 6 in. above high-water level. An open space had been left from top to bottom in the shell-plating of the wrought-iron piers for the insertion of the ends of the girders. Thus the piers and the permanent bracing within them were utilised, along with a temporary "lifting" column of angles in the interior, for

the raising of the span step by step. On the bottom of the girder-work there were temporarily fitted hydraulic rams, one at each corner. The head of the ram cylinder butted against a girder held transversely in the temporary column within the pier by pins inserted through the bracing and angles. The rams, working against this girder, raised the complete span a few inches, when a second temporary girder was pinned into position, again on the "lifting" column, to support the span while the ram receded into its cylinder. The temporary girder on which the ram found its abutment was next raised a distance equal to the stroke of the ram. Thus, step by step, the span was raised to its ultimate position through total heights varying from 47 ft. 6 in. to 68 ft. 6 in. The plates of the piers, temporarily omitted for the insertion of the ends of the main girders, were then completed, and the span secured on its bed-plates.

The bridge was designed to resist a wind pressure of 56 lbs. per square foot. A parapet, 5 ft. high, is built where the trains travel on the top of the girders, in order to prevent the wind from exerting an upward force against the bottom of the carriages. At intervals of 500 ft. in the length of the bridge, provision is made by rocker-bearings for the expansion and contraction of the girders.

The 23,783 tons of iron used is of a strength equal to a breaking tensile strain of 22 tons per square inch, and the 3588 tons of steel in floors, etc., of 27 tons per square inch. There were 37,024 cubic yards of concrete, and 26,419 cubic yards of brickwork built into the foundations, etc. The total cost was £670,000, about £64 per lineal foot. This low cost is in part due to the use of many girders from the old bridge.

The Tower Bridge.

THE Tower Bridge is probably the finest example of bascule structure in the world. Designed by Sir John Wolfe Barry, Bart., and Sir Horace Jones, and built in 1886-1894, it gives a roadway of a minimum width of 49 ft. across the Thames east of London Bridge. The bascules when raised leave a central opening 200 ft. wide for river traffic.

The two open leaves weighing 1200 tons each, are raised and lowered by hydraulic power. Experience has shown that the time taken to open and close the bascules averages five minutes twenty seconds. They are raised on an average twelve times each tide. Even when the bascules are closed the clear height above high-water level is 29 ft. 6 in.—sufficient to allow moderate-sized craft to pass under.

There are three main spans across the river, two of 270 ft. on the suspension principle, with the centre bascule opening of 200 ft. clear. The two river piers, each 70 ft. wide, make up the total length from shore to shore of 880 ft. There are long approach viaducts of granite arching.

The foundations for the river piers were constructed by Sir John Jackson, Ltd., and rest on the clay bed of the river; the load was limited to 4 tons per square foot.[1] There were sunk to a depth of 19 ft. below the

[1] See "Thames Bridges, from the Tower to the Source," by James Dredge, page 1; and "The Tower Bridge: Its History and Construction from the Date of the Earliest Project to the Present Time," by J. E. Tuit, M. Inst. C.E.

river bed eight caissons for each pier, four on each face. The caissons were each about 28 ft. square, and were founded 34 ft. apart. At the ends there were sunk triangular caissons to form the foundations for the cutwaters. The interior of these caissons, and the space between them, was filled with cement concrete. Upon these caissons the piers were built of brickwork with granite facing. Large voids or chambers were left in the piers for the hydraulic machinery, and for the short counterbalancing arm of the bascules. The total cost of the piers was £111,122, equal to £2.37 per cubic yard.

The whole of the steel and iron superstructure was built by Sir William Arrol and Company. Each tower on the piers consists of four octagonal steel columns 120 ft. in height, and each 5 ft. 6 in. in diameter. From a height of 60 ft. above road level to the top the columns are connected together at intervals by girders with heavy diagonal bracing. The towers on the abutments are generally similar to those on the river piers, but are only 44 ft. in height.

The side spans are on the suspension principle. The roadway and footpath decking is carried on longitudinal stringers, borne on transverse girders 61 ft. long and 33 in. deep, which are hung at intervals of 18 ft. by suspension tie-rods from two braced and curved supports, conventionally called "chains," but really heavy curved girders. The lowest part of the chain is not, as is usual, in the centre of the span, but is nearer the abutment to suit the difference in the height of the pier and abutment towers. At each abutment the chains are anchored to massive concrete foundations, while the ends of the chains of both suspension spans at the pier towers are connected by ties some 230 ft. long, stretching between the piers at

The Tower Bridge, with Bascules Closed.

N

the level of the upper footbridges. In this way the loads on the two suspension spans are balanced.

In the bascule span there are two leaves, each carried on a trunnion shaft placed in a recess in the corresponding pier. Each leaf is built up of four longitudinal main girders, 13 ft. 6 in. apart, with bracing and intermediate girders to support the decking. The main girders extend to the rear of the trunnions, where there are formed chambers which carry about 350 tons of counterweight for balancing each leaf on its turning shaft.

The hydraulic machinery for opening and closing the bascules is located in a chamber in the pier. This machinery operates, through gearing, a quadrant on the girders of the bascule, which is thus tilted from the horizontal to the vertical position for opening—or backwards to the horizontal for closing—the passage for ships. The engraving on the opposite page shows the bascules open.

In the bascule span, high above the main level of the roadway, there are two foot-bridges. To these access is provided by means of stairways and hydraulic lifts within the towers. These foot-bridges have a span of about 230 ft.; they are 141 ft. above high-water level. They are quite independent of each other, and each has a footway 12 ft. wide. The bascule is, however, so quickly opened and closed for the passage of river traffic, that the delay to pedestrians, as well as to vehicular traffic, is negligible, and the high-level bridges are never used.

In the construction of the bridge there were used 235,000 cubic feet of granite and other stone, 20,000 tons of cement, 70,000 cubic yards of concrete, 31,000,000 bricks, and 14,000 tons of iron and steel. The total cost of the bridge was £830,000.

We are indebted to Mr. John Gass, the engineer at

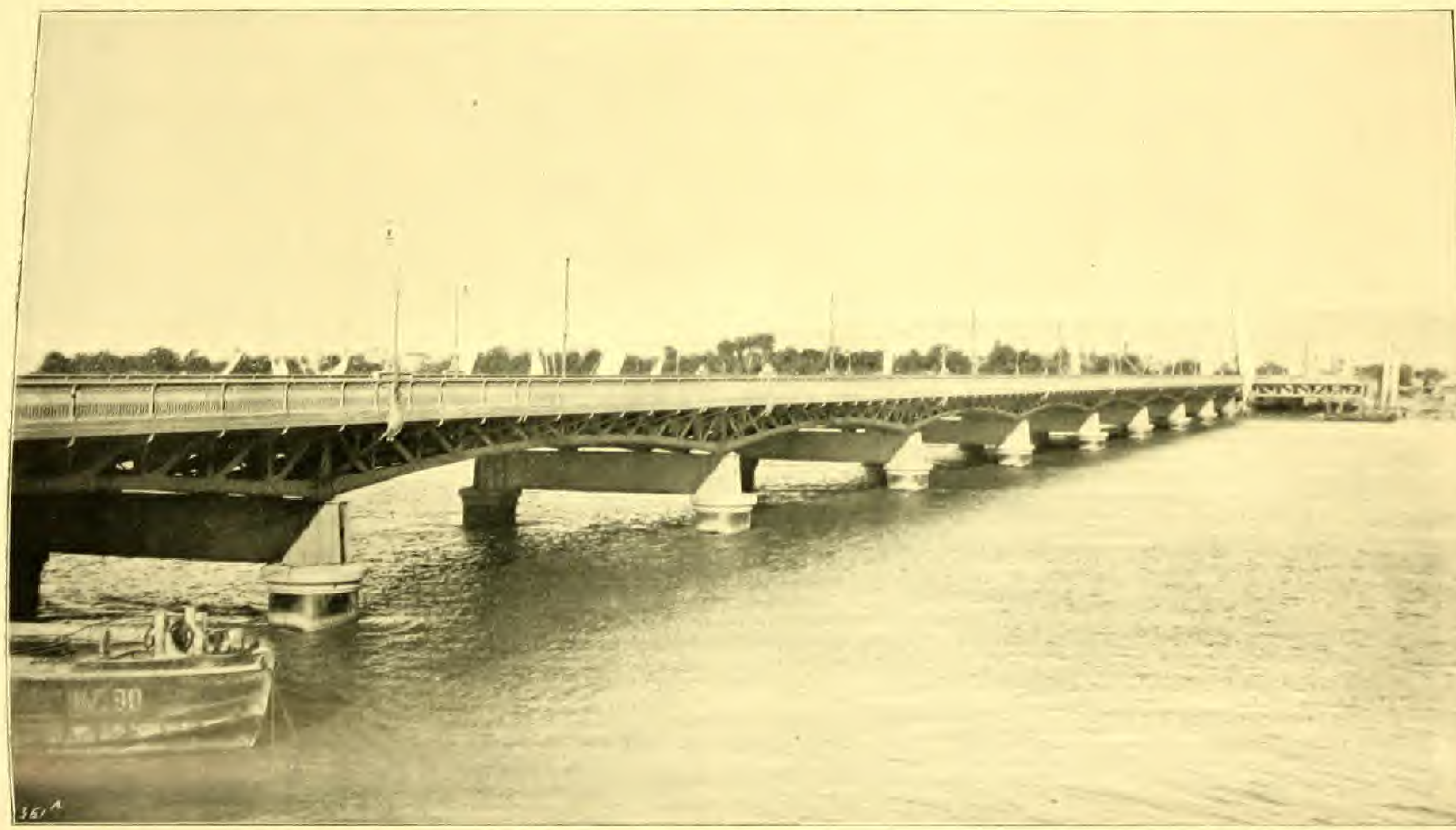
The Tower Bridge, with Bascules Raised.

the bridge, for the following particulars relating to the working of the bascule span :

	seconds.
Time taken to clear the traffic off the bascules	30
Time taken to withdraw the locking bolts	25
Time taken to lift the bascules through an angle of 82 deg. ...	90
Time taken to allow vessel or vessels to pass	various
Time taken to lower the bascules	90
Time taken to shoot the locking bolts and resume traffic ...	25

The average delay to the vehicular traffic per lift was 5.5 minutes in 1899, 5.25 in 1900, 5.22 in 1901, 5.33 in 1902, 5.07 in 1903, 5.00 in 1904, and 5.00 in 1905, and it has since then continued at this period. The average number of openings of the bascules per day of twenty-four hours was 22.41 in 1900; 23.71 in 1901; 25.23 in 1902; 25.10 in 1903; 24.27 in 1904; and 23.78 in 1905. During the first year after the opening of the bridge the average number of openings per day was 17. The number of openings on one day has reached a maximum of 55; this has occurred on two occasions.

Under ordinary conditions an average of 87 horse-power is exerted by the hydraulic engines in opening and closing each bascule.



The Bridge across the Nile from the Island of Rodah to Ghizeh.

Road Bridges over the Nile at Cairo.

IN the summer of 1903 the Egyptian Government issued specifications inviting tenders and designs for three road bridges to be constructed over the Nile at Cairo.[1]

For each bridge thirty-eight designs and tenders were received from thirteen firms, six of whom were French, one each German, Belgian, Italian and Swiss, and three British, one of the latter submitting an American design.

A Commission was appointed to examine the designs and calculations and report to the Government. After a careful and prolonged examination of the various proposals, the designs and tender of the joint firms of Sir William Arrol and Company, Limited, of Glasgow, and Messrs. Head, Wrightson and Company, Limited, of Middlesbrough, were accepted for constructing the three bridges. The successful designs were prepared by the Civil Engineering Staff of Sir William Arrol and Company, Limited.

The principal bridge spans the Nile from Ghizeh, near the road to the Pyramids, to the Island of Rodah, and the two smaller bridges connect Rodah Island with the main road leading to Old Cairo.

A double line of tramway of one metre gauge passes over all the three bridges, and connects up the Cairo lines with the one to the Pyramids.

The position of the main bridge required careful

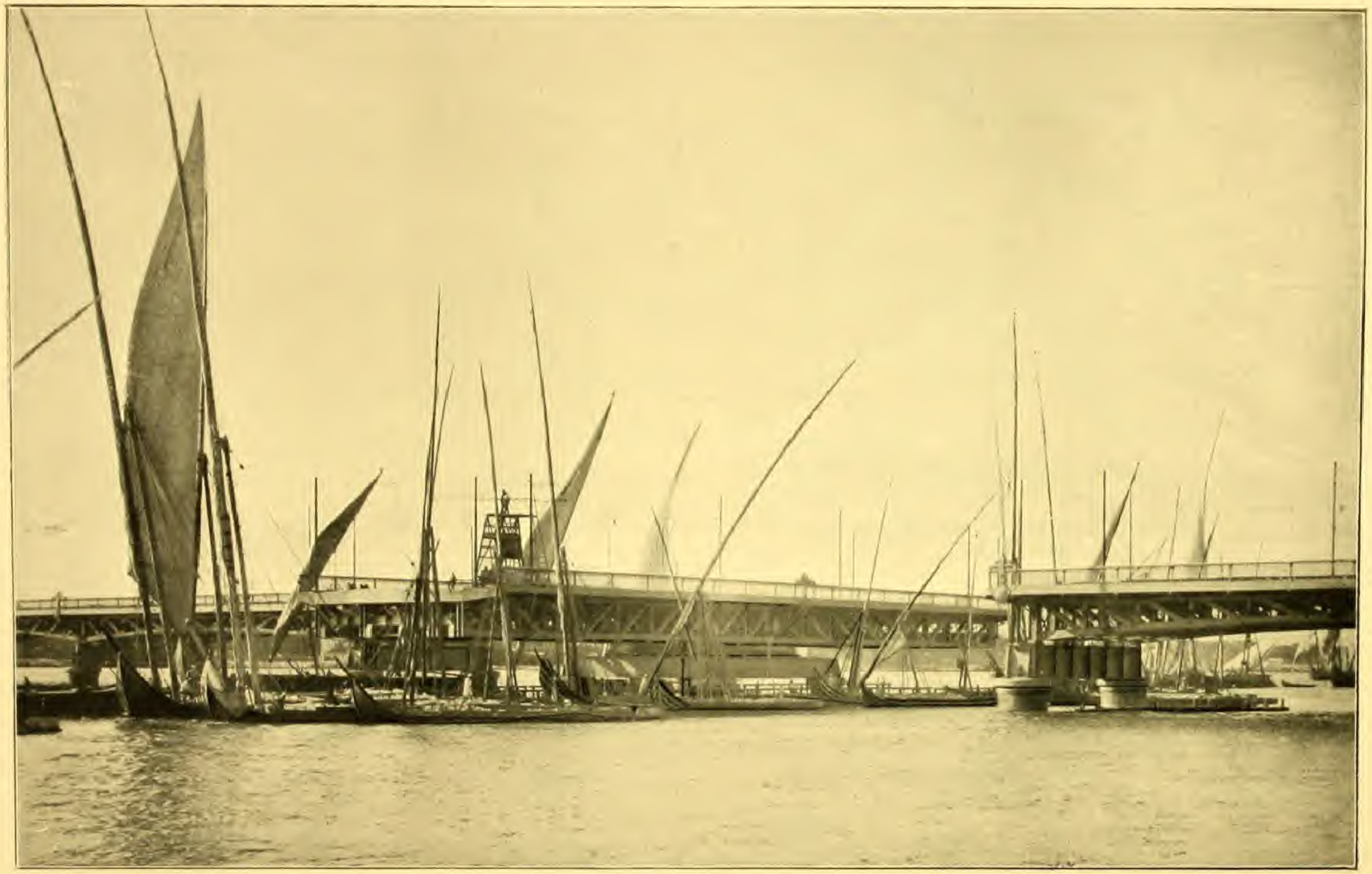
[1] See ENGINEERING, vol. lxxvii., page 682; vol. lxxxi., page 42; vol. lxxxii., page 483; vol. lxxxiv., page 342; vol. lxxxv., page 40.

attention to be given to the æsthetic effect of the design, a point too often forgotten in designing such structures. It was considered that many people would regard the larger bridge not merely as a means of crossing the river, but as offering special facilities to enjoy the scenery and cooler atmosphere in the vicinity of the Nile, and consequently it was most desirable that no portion of the structure should in any way interrupt a clear view of the river and city of Cairo.

The masonry abutments at the entrance to the main bridge have substantial masonry pilasters of an Egyptian character, designed to be in accordance with the dignity of the undertaking. Provision is made that suitable statuary may be placed on the pilasters if considered desirable by the Egyptian Government. The principle adopted throughout the design was to adhere to a form which was structurally correct, while adopting an architectural treatment which would emphasise the construction and give suitable expression to the structure. The general effect is pleasing, and when viewed from the river banks, the bridge has a graceful appearance.

The main bridge is 1755 ft., or about one-third of a mile, in length between abutments, divided into ten spans of 140 ft., two end spans of 70 ft., and a double-swing span 220 ft. in length with two clear openings of 66 ft. The bridge is 66 ft. in width, between parapets, divided into two footpaths 8 ft. wide, and a roadway of 50 ft. in width.

The piers of the bridge are formed of steel cylinders, 18 ft. in diameter at the base, filled with concrete. The load at the bottom of each cylinder is about 1400 tons, making a total of nearly 40,000 tons on the foundations. Over 7000 tons of steel and iron were used in the work.



The Swing Span Open for the Passage of Boats.

One of the Two Bridges between Rodah Island and Cairo.

The Main Bridge across the Nile with its Test Load.

The electrically-operated swing span, illustrated on the Plate facing page 94, is 220 ft. in length, weighs nearly 1000 tons, and is opened or closed in three minutes, including all operations. Provision is made against an interruption of the electric current, and two men can open the bridge by manual power.

To found the cylinder piers it was necessary to remove about 16,000 cubic yards of grey sand, and this was done under air pressure. The foundation work was completed one month ahead of time, notwithstanding some early delays in commencing operations. The whole of the sinking was done by native labour under European supervision.

Considerable difficulty was experienced in getting labour for the work of building and riveting the steel superstructure. The general prosperity of the country caused suitable native labour to be scarce, and such as could be obtained had to be specially trained under European foremen to work the pneumatic and hydraulic riveting machines. Over half-a-million rivets had to be put in at the site, and great credit is due to the staff for the careful supervision exercised, as the total percentage of defective rivets which had to be replaced was almost the same as obtains at home with skilled white labour. With regard to this point, the late Sir Benjamin Baker reported to the Egyptian Government that the work done by the native riveters was of a satisfactory quality.

The erection of the whole of the steel superstructure was completed fully two months ahead of the forecasted time of the programme made out at the beginning of the work.

The bridges were officially tested by the Government with most satisfactory results. The various operations in

connection with the opening span were carried out well within the specified time. The construction of this bridge is most creditable to the joint contractors, Messrs. Sir William Arrol and Company, Limited, and Messrs. Head, Wrightson and Company, Limited. In support of this statement it is only necessary to quote from the report of the late Sir Benjamin Baker, in which he states: "I have to-day, February 1st, 1907, inspected the above bridge, and am able to certify that as regards material and workmanship it is entirely satisfactory, and that in all respects the structure is a fine example of the high class of work which can now be turned out by bridge builders of great experience, whose manufactories are supplied with all the most modern appliances."

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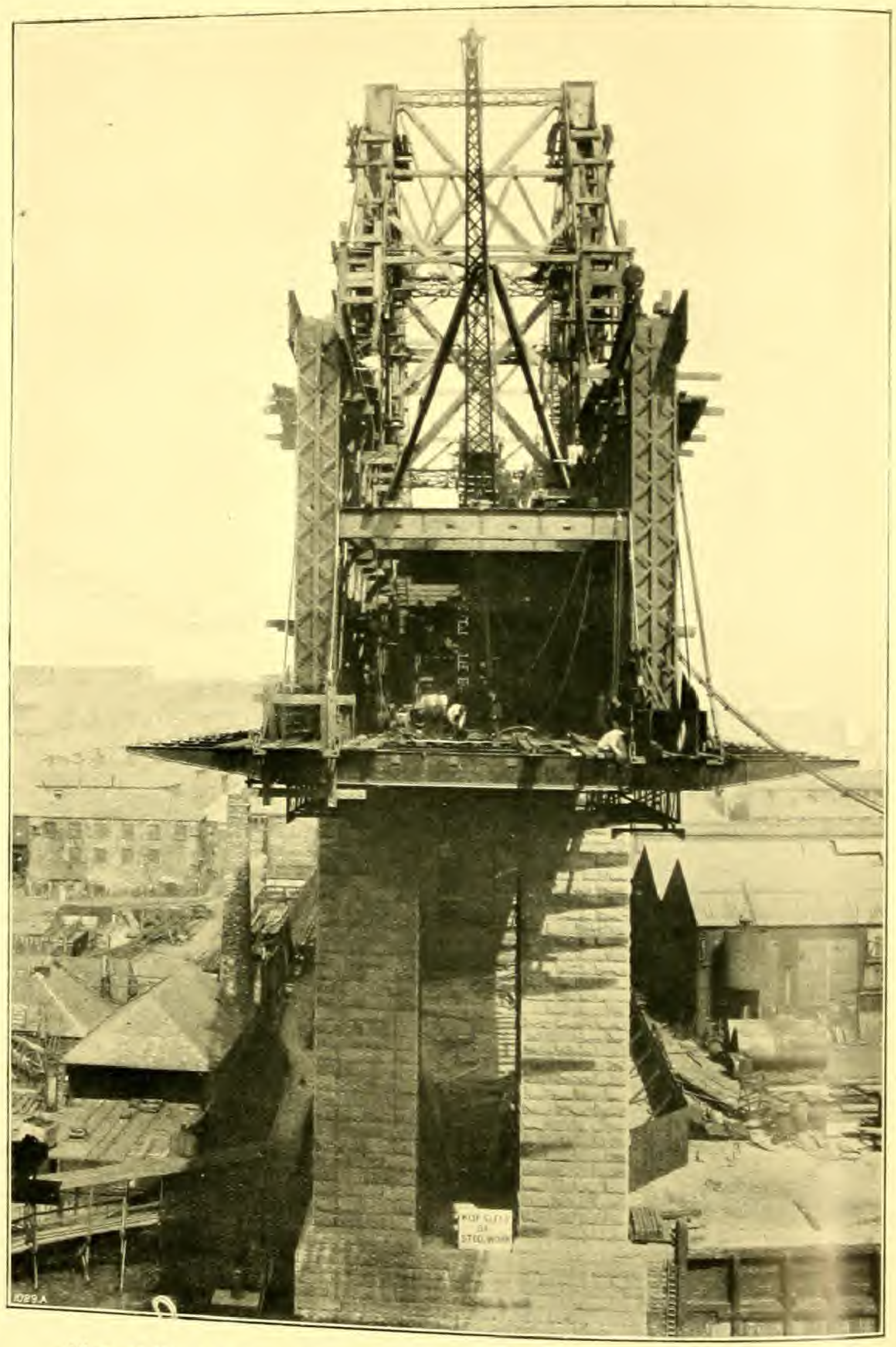


View of Wear Bridge, during Erection, showing Pier and Double Decks.

Bridge over the River Wear at Sunderland.

THE new double-decked railway and road bridge over the River Wear at Sunderland,[1] with railway and roadway approaches, was constructed from the designs of Mr. Chas. A. Harrison, M. Inst. C.E. The bridge and approaches have a total length of about $1\frac{3}{4}$ miles, and they formed one of the largest bridge contracts let in this country since the Forth and Tay Bridges, which were built by the same firm. The contract included the whole of the works for a double-line railway throughout, and for the street approaches and the roadways over the bridge. The main bridge has two decks, the upper deck carrying a double-line railway track for the North-Eastern Railway Company, while a roadway and footways are carried on the lower deck and provide communication across the river between Sunderland on the south side of the river and Southwick. About 8500 tons of permanent steelwork was required in the construction of the bridge, as well as 500,000 cubic feet of granite, 60,000 tons of red sandstone from the Locharbriggs Quarries, near Dumfries, one third of a million bricks, and about 300,000 cubic yards of spoil in the banks and cuttings.

The general design of the bridge is shown on the illustration opposite. The total length of the bridge proper is 1560 ft. The northern approach between the railway

[1] See ENGINEERING, vol. lxxxiv., page 43; vol. lxxxvi., page 533.

embankment and the north abutment is formed of seven stone arches carrying the roadway and footways, and supporting the steel trestles for the railway tracks. The railway and roadway approach each other at an angle, and intersect on the north abutment. Between the north and south abutments are two land spans of 200 ft. on the north side of the river, a river span of 330 ft. clear, and a land span of 200 ft. on the south side of the river. From the south abutment to the embankment the roadway is carried upon stone arches surmounted by steel trestles for the railway tracks as on the northern approach.

The foundation for the main river pier was sunk, under air pressure, to 75 ft. below high water by means of an immense rectangular caisson, 63 ft. long by 35 ft. wide. The caisson was filled with concrete to facilitate sinking. A large chamber, open at the bottom, was left in the caisson so that the men could work inside and excavate the soil, which was removed through shafts in the roof of the chamber and passed through air locks to the open. When the caisson reached the final depth, the chamber was completely filled with concrete, so as to provide a large surface to distribute the great weight to be borne on the foundation. The usual temporary caisson or cofferdam above the permanent shoe was dispensed with in sinking these piers. The masonry of the pier was built up above high water as the sinking proceeded. After the sinking was completed, the granite piers were continued up to the girder seats about 85 ft. above high-water level. The total weight of each completed pier is about 16,000 tons, and it carries on the top of it a total load of 4300 tons.

The land spans consist of two parallel main girders, 224 ft. long and 30 ft. deep, placed 32 ft. apart, with a suitable floor suspended from the horizontal bottom

members. The railway floor is placed between the main girders, so as to allow a clear headroom of 18 ft. above the roadway. Each land span weighs about 1000 tons. They were built in their place on staging formed of three timber piers, with suitable girders on their tops to support the weight of the span.

The river span illustrated on the following pages is formed of two main girders, and is 353 ft. long and 42 ft. deep in the centre of the span. The road and railway floors are similar to those for the land spans. The weight of steelwork in this span is 2600 tons.

The erection of the span over the river required considerable engineering ingenuity and judgment on account of the exceptional difficulties to be overcome. On each side of the river are shipbuilding yards, from which the ships are launched into the water immediately under the bridge. In the Act of Parliament permitting the construction of the bridge it was expressly stipulated that the river was to be kept clear at all times for navigation. It was left free to the contractors to erect the bridge in any manner which would not encroach on the navigable waterway. To put staging in the river upon which to build this span was clearly out of consideration.

After consideration of various schemes, Sir William Arrol and Co. decided to convert the river span temporarily into cantilevers and build it out from each pier, piece by piece. In buildi g the central girder between the projecting cantilevers of the Forth Bridge a similar method was adopted, but although the central girder of the Forth Bridge and the river span of the Wear Bridge are of exactly the same length, the weight of the Forth Bridge girder was only 850 tons, or one-third of that for the River Wear. The valuable experience gained by Sir William Arrol and

Company, Limited, at the Forth Bridge was of especial value when dealing with the greater problems presented in the erection of the river span of the Wear Bridge.

The first portion of the span was built upon the pier, and the top of the girders tied back to the end posts of the land span which rested on the same pier. These ties were of sufficient strength to enable about 75 ft. of the river span to be built out from the pier. While this portion was being built, a steel tower 70 ft. high was erected upon the portion of the river span immediately above each pier, and the top was tied back to the farther end of the adjoining land span. From the top of each tower steel ties were brought down and connected to the girder at points about 70 ft. from the pier, which allowed the river span to be built a further 48 ft. from each pier, where other ties from the top of each tower were connected to the bottom members of the main girders. When these ties were connected the river span was built out to the centre of the river, 170 ft. from the piers. At this point careful measurements were made and sent to Glasgow, where the closing lengths were made and forwarded to the site.

The placing of the closing lengths in position, and the riveting of them up, was an operation requiring great care and judgment. Variations of temperature caused the projecting ends of the girders to rise and fall from $\frac{1}{2}$ in. to $\frac{3}{4}$ in., and to approach and recede from each other from $\frac{1}{4}$ in. to $\frac{1}{2}$ in., while the sun caused the ends to move westwards as much as $1\frac{1}{2}$ in. in the morning and again eastwards in the afternoon. The closing measurements were made on a dull day, when the temperature was uniform, and only under the same conditions could the closing lengths fit their places. To be independent of the variations of temperature, special provisions were made.

First Stage in the Building of the River Span.

Final Stage in the Building of the River Span

At the ends of the land spans an hydraulic arrangement was fitted to push or pull the girders horizontally, and under the ends of these spans hydraulic jacks were placed to raise them, and thereby vary the levels at the centre of the river span. These special provisions, however, were not required, as the ends came together accurately under normal temperature conditions.

When the riveting of the closing lengths was completed, the temporary ties were relieved of stress, to permit of their removal, by raising the ends of the land spans a sufficient height to allow all elongation in the ties to be given up, and the main girders of the river span to settle down under the reversal of stress in the booms.

The magnitude of the task is evident from the fact that about 800 tons of steelwork was employed in the temporary work for erecting this span. It was realised by Sir William Arrol and Company, Limited, that enormous risks were being undertaken, and the greatest care was given to the temporary work to get an erection scheme of safe, substantial, and economical character. The plant and temporary steelwork for the erection was accurately made. All the joints in the temporary ties and towers were made with turned steel bolts of a hard driving fit, accurately turned to gauges, and fitted by mechanics. About 20,000 bolts were used, and several tests of their shearing strength were made under circumstances similar to their working conditions. Some idea of the enormous forces dealt with may be had from the knowledge that the stresses in the back ties of each half span amounted to about 1200 tons, and in the front ties to 1400 tons. The maximum weight suspended from the temporary ties before the closing lengths were fixed was 2400 tons. To ensure that the temporary ties took the

portion of the load for which they were designed, an hydraulic stressing gear of unique design, capable of exerting a force of 800 tons, was employed in putting an initial stress in each tie. This gear was tested to 1200 tons before leaving the contractor's works. The hydraulic screw jacks are illustrated on this page. The ram had a screw thread upon it, and engaged with a nut where it projected through the cylinder. As the ram was forced out of the cylinder by the hydraulic pressure, the nut was kept continually screwed back to maintain constant contact with the top of the cylinder. In the event of failure of the hydraulic pressure, it was impossible for the ram to run back, and it could be held in any desired position for any length of time to allow any operations to be carried out.

Hydraulic Stressing Gear on Temporary Ties.

The Scherzer Rolling-Lift Bridge Over the Swale.

THE engravings on pages 105 and 107 illustrate the Scherzer Rolling-Lift Bridge, which carries the main line of the South-Eastern and Chatham Railway, and the public highway, across the Swale, in the County of Kent.[1] The span is 62 ft., and the total width of the bridge between the centres of the girders is 33 ft. The total weight of metal in the bridge is 962 tons.

The bridge which it replaces was a double bascule, operated by hand-power, and built about 1862. The new structure was designed by the late Sir Benjamin Baker, K.C.B., who decided to adopt the Scherzer type, in which the rolling weight of the bascule is balanced and heavy gearing dispensed with, whereby friction is minimised. Thus on the official trials the whole span, weighing 520 tons, was opened in fifty seconds, with the motor developing only 9 brake horse-power.

Great interest is associated with the foundations, as the old piers had to be underpinned. The cutwater ends of the old piers were first taken away, to enable new steel caissons to be sunk for four new piers. These are founded at a greater depth than the cast-iron piles, which constituted a feature of the old piers. The old bridge foundation was

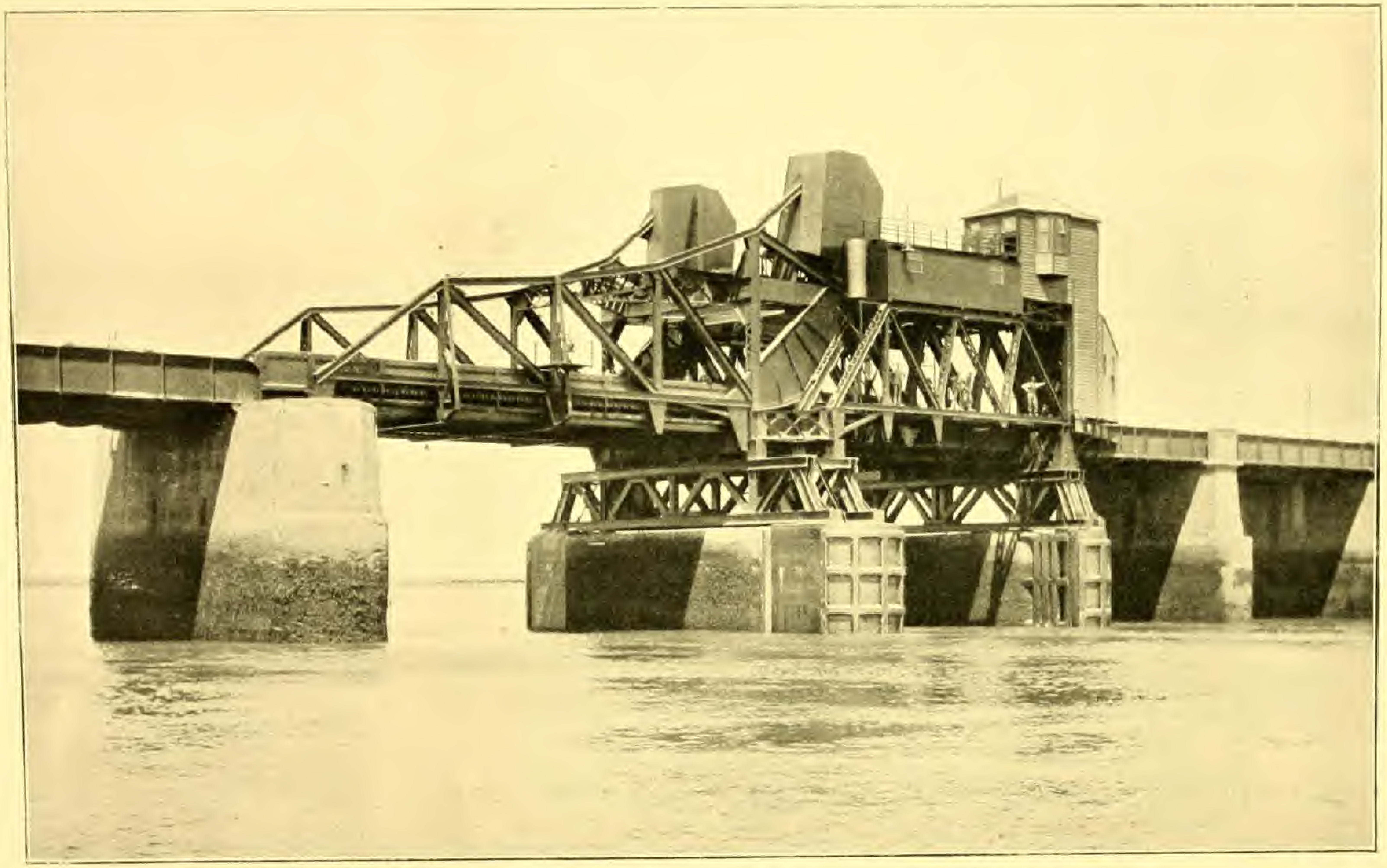
[1] ENGINEERING, vol. lxxix., page 762.

next underpinned with timbers, and steel beams were passed through the heart of the old brick piers, which were hollow. The upper walls of the old piers were then partly cut away for a sufficient depth, to allow heavily-braced cross-girders to be put in position between each pair of the new piers. These girders carry the new superstructure.

The Scherzer system also offered the special advantage that erection could proceed without interference with the traffic either of the railway or the highway. The rolling span was built and riveted together upon a platform over the old structure, and was subsequently lowered into position within the short period when traffic had ceased between Saturday night and Monday morning. The staging was erected over the old bascule span and the approach span immediately behind it. It was supported on steel girders and posts, and on it were carried also the necessary cranes, etc., for the construction of the new Scherzer span. The railway and highway beneath were thus entirely free for the passage of traffic during the progress of construction of the new span.

For the tilting mechanism of the rolling Scherzer span, there are four strong braced girders outside of, and parallel with, the old girders of the approach span, which was retained. The new longitudinals carry track girders with a planed flange and projecting teeth, in which engage the geared segmental girder of the Scherzer lift span, well shown in the engraving on the opposite page. The circumferential surface and teeth were milled.

The lifting span, 65 ft. long between centres of bearings, had been built and fitted temporarily at the Dalmarnock Works of the Company, and the parts duly marked were forwarded for final erection and riveting on the site. This span is composed of two outer braced girders, and a centre



The Scherzer Rolling-Lift Bridge over the Swale: Closed.

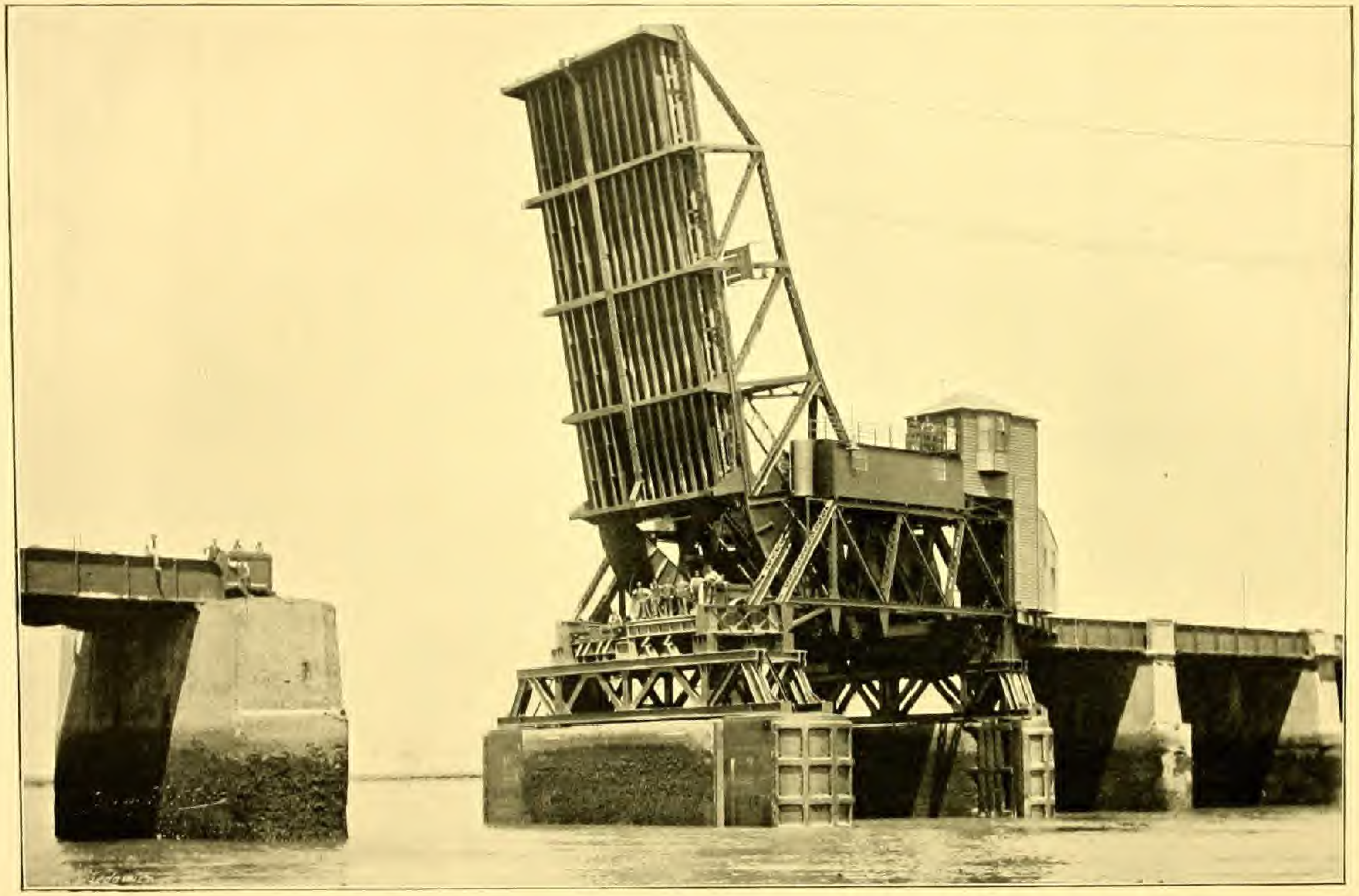
plate web girder, which serves to divide the roadway from the railway. The floor surface is of steel plating, covered with asphalte on the railway side, and planked for the roadway. This is illustrated in the view on the opposite page, showing the rolling-lift span open for the passage of river traffic. The single line of railway crossing the bridge is attached to longitudinal timbers laid in troughs.

The segmental girders curving backwards from the end of the posts at the rear of the Scherzer span carry on their upper terminations counterweight boxes, containing a sufficient weight of metal to balance the lifting span; and this kentledge is so disposed as to be in exact counterpoise with the span in any position.

Carried on the approach trusses over the roadway and railway are the platform and houses for the accommodation of the operating gear. The dynamo, driven by an oil engine of 9½ brake horse-power, supplies storage batteries from which the motors take their current. There is also an emergency hand-gear for lifting and lowering the span.

Alongside the engine-house there is a signal cabin, from which are controlled the operating machinery, the signals, the locking gear, the gates on the railway and roadway, etc.

Notwithstanding the difficult work in connection with the foundations, the total cost of the bridge was only £38,500.



The Scherzer Rolling-Lift Bridge over the Swale: Opened.

Viaducts Over the Rivers Barrow and Suir in Ireland.

THE next typical structures which we select for description are viaducts over the Rivers Barrow and Suir, near Waterford, Ireland. Both are interesting; the first-named is the longest viaduct in Ireland. The two bridges constitute important links in the chain of communication inaugurated in 1906 as a new route between London and the South and West of Ireland.

The passenger travels over the Great Western Railway from the Metropolis to the new harbour of Fishguard, in Wales, a distance of 262 miles. Thence there is a day and night service across St. George's Channel to a new harbour at Rosslare, in Ireland, 54 nautical miles distant, by three new turbine-driven steamers, with a speed of $22\frac{1}{2}$ knots.[1] From Rosslare a new line, 38 miles in length, has been constructed to Waterford, joining there with the system of the Great Southern and Western Railway of Ireland, communicating with Killarney and other tourist districts. The new route shortens by 100 miles the journey between London and some of the most beautiful districts in Ireland.

The specially notable features of the works in Ireland were the long viaducts across the rivers Barrow and Suir, the former on the new main route, and the latter linking

[1] See ENGINEERING, vol. lxxx, page 178; vol. lxxxii, page 106.

the new line with the existing Waterford, Dungarvan, and Cork line of the Great Southern and Western system. For the new works Sir Benjamin Baker, K.C.B., was chief engineer, in collaboration with Mr. James Otway, one of the engineers of the Great Southern and Western Railway of Ireland.

The Barrow is a broad river, with a relatively narrow navigable channel of great depth — 39 ft. 9 in. at low water — on the Kilkenny side. The Suir is somewhat similar, and both rivers join where they enter the broad estuary which constitutes Waterford Harbour. The River Barrow is crossed by a single line of railway, 5-ft. 3-in. gauge, 100 yards above its confluence with the Suir, and about 6 miles from the town of Waterford; while the Suir is spanned 1 mile above Waterford.

The superstructures of both bridges are alike. The Barrow Bridge is 2131 ft. long between the faces of the abutments, and consists of thirteen fixed spans, with a swing-span over the river, giving a passage 80 ft. clear for the traffic on each side of the centre dolphin. The Suir Bridge is 1205 ft. in length, also between abutments, and includes six spans of 148 ft., one of 133 ft., one of 102 ft. 9 in., and an opening span—in this case of the Scherzer rolling-lift type—of 50 ft. in the clear.

Interesting timber work was involved at the Barrow Viaduct, not only in the forming of the temporary staging for the sinking of the piers, but also in the construction of the swing-span dolphin. This latter extends for a distance of 150 ft. in line with the river, and is 39 ft. in width at the centre, tapering to 25 ft. at the ends.[1]

In the construction of the bridge interest was largely centred in the piers, as several of these had to be

[1] Engineering, vol. lxxxi., pages 673, 716, 780, and 841.

sunk to great depths, the maximum being 117 ft. below high-water level. The air pressure reached 43.5 lbs. per square inch. On pages 16 to 18 we have described the procedure and illustrated the piers; while on page 20 details are given of the compressed air-locks.

The two cylinders forming each of the piers are braced at the top by cross capsill girders, which form the seating for the main longitudinal girders. These are 20 ft. deep over angles, and are spaced at 16 ft. 6 in. centres. Each girder is constructed in eight bays, designed so that no rain water may lodge in any part.

The girders were built in sections at the Glasgow works, and despatched to the site, where they were erected in position on wooden trestles placed on the temporary staging, the complete span being thus put together ready for lowering on to the bearings. The cross girders carrying the permanent way are at from 18 ft. to 19 ft. 3 in. centres. The bottom lateral bracing consists of angles riveted to gusset plates at the base of the vertical posts of the main girders. The lattice bracing on top is clearly illustrated in the view on the opposite page. The portal bracing there shown was erected at the ends of each span, and forms the terminal member of the system of top lateral bracing; it ties together, and forms the strap between, the upper part of the raking-posts.

We turn now to the pivot piers and the swing span, which, with its moving parts and live ring, weighs 303 tons. The pivot pier for the swing span consists of four cylinders, braced, on both the transverse and longitudinal lines, by ties extending 20 ft. below water level.

The girder framework, for carrying the roller path, the pivot, and the live rollers of the swing span was built upon the main capsill girders resting on the top

of the four piers. These form a rectangle supporting a framework riveted and set perfectly level in position, before the concrete filling of the cylinders was put in, prior to

The Viaduct over the River Barrow.

the bed-stones of the main capsill girders being grouted up. The support for the circular track for the live roller-ring was thus made perfectly true to level. The concreting was then completed, and its surface coated with asphalte.

At the centre of this framework there is a large

foundation plate, octagonal in plan, on which the pivot is bedded and held in position by bolts. The holes for these bolts were bored on the site after the pivot had been set in position, as great care had to be taken to see that the pivot was dead true in the centre.

The live roller-ring is carried on this girder framework. The engraving on page 7 gives a clear idea of this ring, and of the centre pivot round which it rotates. The pivot is of cast steel, and affords a bearing around the complete circumference for supporting a ring-plate, to which are connected the radial rods extending to the rollers. On each rod there is an arrangement for adjusting its length. The rollers are of wrought steel.

On the tops of the rollers a circular girder was built. This is known as the "drum" girder, and to the underside of it there is bolted the cast-steel tread resting on the live rollers. The drum girder, which is 23 ft. 4 in. in diameter, thus constitutes a circular support transmitting the weight of the swing span to the top of the live rollers. A series of girders connect this circular support with a central framework carrying the steel pivot ring. This ring conveys no weight to the pivot, but transfers to it all lateral pressure.

The swing span is 214 ft. 6 in. long. In the top boom of the centre panel of the girders there are transverse members supporting the operating machinery room, which is thus over the railway track. On a still higher level is the look-out cabin, in which there are located the levers, not only for the signalling gear, but for operating the bridge. Telegraphic and telephonic connection is made with the land on each side of the bridge.

The motive power for operating the swing span is electric, with emergency hand-gear. The power equipment,

General View of the Viaduct over the River Barrow.

located in a power-house on the dolphin, consists of two $9\frac{1}{2}$ brake horse-power oil engines driving dynamos generating electric power at 150 volts, which is stored in 61 cells of the Tudor make. The time taken to fully charge these cells, when both engines are running, is six hours; the fuel consumption is seven to eight gallons of oil.

The contract condition that the swing span should be opened and closed in two minutes was fulfilled without difficulty. The weight to be turned, including the live ring, is 303 tons. Two turning motors in the machine-room, each of 20 brake horse-power, singly or together rotate the main horizontal shaft, through friction clutches adjusted to a maximum transmission of 20 brake horse-power each. From this main shaft the power is transmitted by bevel wheels to two vertical shafts at opposite corners of the structure. These connect by spur gearing at their lower ends with the driving shafts attached to the drum girder. The last-named shafts carry the driving pinions, which engage with the rack fixed to the top of the pivot. The turning of the rack swings the opening span.

There are also provided electrically-driven lifting jacks at each end of the swing span, to lift it from its end bearings before it is swung. There are locks to secure the bridge in its seating when closed.

The Caledonian Bridge Over the River Clyde at Glasgow.

THIS bridge is one of the broadest and most substantial structures yet built, carrying nine lines of rails across the River Clyde into the new Central Station at Glasgow of the Caledonian Railway Company.

The new structure was built in 1904-5 alongside an earlier bridge, also constructed by the firm in 1875-78 (see page 4). It is part of extensive reconstruction work at the station, necessitated by the great increase in traffic into this Glasgow terminus.

The new bridge was designed by Mr. Donald A. Matheson, M. Inst. C.E., the chief engineer of the Caledonian Railway Company, and is remarkable no less for its great strength than for its economy in construction, the cost working out at about £3 per square foot of decking. The river is about 410 ft. wide; the bridge, with the adjacent street spans, has a length of 752 ft., and an average width of 120 ft. The weight of steel work employed in its construction totalled 11,000 tons. The principal spans are 160 ft., 200 ft., and 178 ft.

For forming the foundations, steel caissons were sunk under compressed air: six for the shore piers, and two for each of the river piers. These latter were sunk to a depth of over 80 ft. below high-water level, into hard sand and gravel. The piers which are built on these foundations

are of red brickwork, faced with blue brick up to near the low-water level, and with light granite thence to the cope.

The superstructure of the river spans is of steel lattice-girders; there are ten girders in the width of the bridge, and the spacing between them increases as the bridge fans out towards the station on the north side of the river. The spans over the streets at each end of the bridge are of web girders. The flooring, of Hobson's trough type

View along the Bridge.

on the river spans and of buckle plates on the street spans, is on the top of the main girders; this permitted points and crossings to be introduced wherever desirable.

An ornamental lattice parapet, with cast-iron cornice, cope, and rosettes, is carried out from the outer main girders on ornamental steel brackets, and gives a graceful appearance to the bridge, as shown on the engraving on the opposite page. The brackets carry a footpath fully 5 ft. wide along the whole length of the bridge, for the use of workmen and officials.

The Caledonian Bridge over the River Clyde at Glasgow.

Sir William Arrol and Company, Limited, were also entrusted with the raising of the old bridge by nearly 3 ft., to coincide with the level of the new bridge and of the greatly enlarged Central Station. In the old bridge there are five spans, varying from 200 ft. to 163 ft.; with two shore spans 70 ft. and 100 ft. respectively. There are two main girders in each span, at 50-ft. centres. The floor is carried by cross girders resting on the bottom booms of the main girders. The main girders in the river spans are braced overhead.

As soon as traffic could be transferred to the new bridge the work of raising the old structure was commenced. This was early in October, 1905, and the work was completed and the bridge reopened in April, 1906. The first span raised was the centre one, of 200 ft. in length and 800 tons in weight. Large plate brackets were secured with turned bolts at each end of the two main girders, and two hydraulic jacks were placed under each bracket. The eight jacks were connected to the same pump, which worked at a pressure of 1800 lbs. per square inch. Hardwood packings were inserted from time to time between the masonry of the pier and the bearing-plates, so that at no time was the space between the bearing-plate and the packing allowed to be greater than one quarter of an inch. The lifting was continued about 3 in. higher than the ultimate level, in order to allow a stool to be put in place underneath the bearing plate. These stools were built of steel plates and angles, planed at top and bottom and brought to the site finished. When put in place, the span was lowered upon them. The lifting brackets were then taken off, and bolted on to the girders of the next span to be raised.

The Redheugh Bridge over the River Tyne.

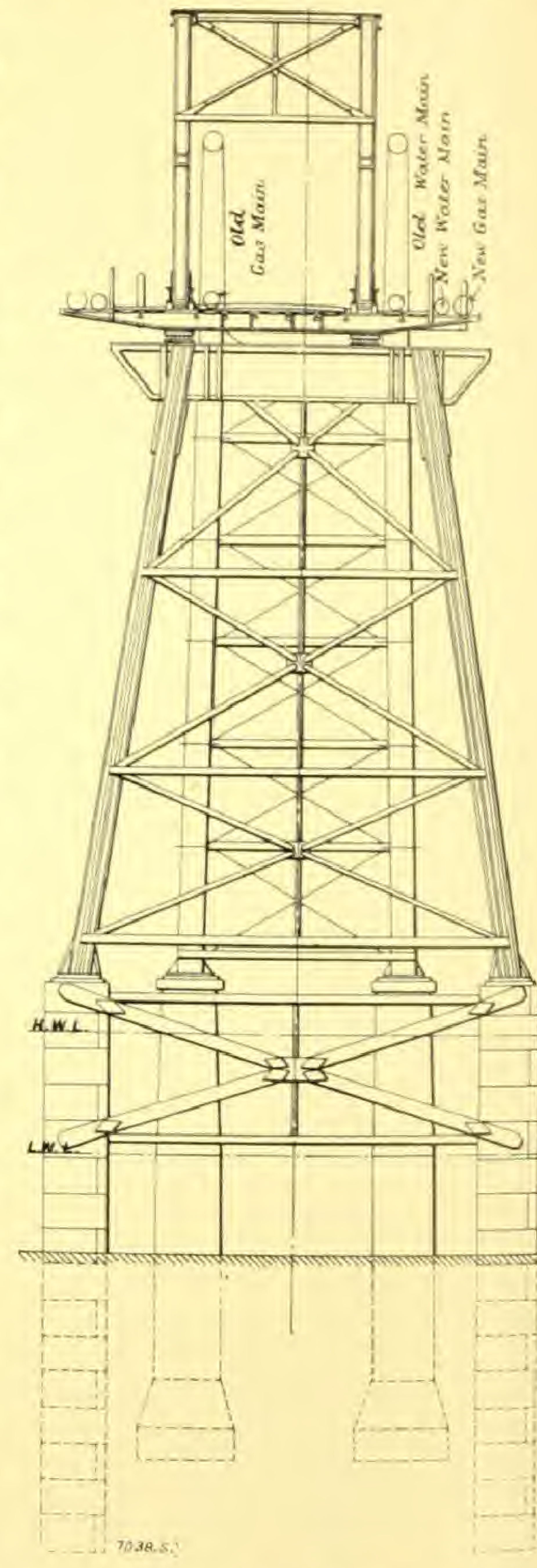
THIS bridge,[1] of which an engraving is given on page 15, was designed by Messrs. Sandeman and Moncrieff, M.M. Inst. C. E., of Newcastle-on-Tyne, and carries the road traffic between Newcastle and Gateshead across the River Tyne, at a point where the waterway is about 850 ft. wide. The problem was to build a bridge requiring deep foundations and long spans on the same site as the old structure—which was not in a safe condition—without interfering with the traffic.

As shown in the section of old and new structures on the next page, the old bridge was supported on cast-iron piers, and the superstructure was built up of braced girders having tubular booms, which were used originally as mains for the passage of gas and water across the river. Preparatory to the building of the new bridge, large timber trestles were constructed to protect the old piers, and to temporarily support the old girders during the process of constructing the new bridge. At the same time, extreme care had to be exercised to prevent settlement during the sinking of the caissons for the new piers.

While new approaches were being made of girder work carried on masonry piers, with a large abutment pier

[1] See ENGINEERING, vol. lxxii., pages 550, 644.

on each bank of the river, the difficult work of sinking the new foundations was proceeded with. For the four spans across the river three piers were made, each composed of four cylindrical caissons sunk to a depth of 65 ft. below high-water level, into a firm bed of shale. These caissons were filled with concrete, and strongly braced in pairs. Steel columns were built upon the caissons, with an inward batter towards the top, where a strong platform of girders was constructed, uniting together the four columns in each pier. These girders, of lattice work, measured 48 ft. over all, and carried the main longitudinal members of the bridge. These latter were built as cantilevers in both directions, until a junction was formed with the girder work from the neighbouring pier. The girders are of lattice construction, with N bracing and intermediate struts and posts. The two spans adjacent to the shore are about 168 ft., and the two river spans 248 ft. each. The longer girders are 36 ft. deep, and the shorter 26 ft. 10 in. deep. For each span there are two main longitudinal girders, spaced 23 ft. centres apart.



Section showing Old and New Redheugh Bridges.

Redheugh Bridge.

As the girders have raking, instead of vertical, end posts (see engraving on pages 15 and 121), it was necessary to temporarily continue the upper chords over the piers, and this was done by means of ties in which toggle gear was introduced. These temporary ties were kept in tension owing to the overhang of the girders when in course of construction as cantilevers. The toggle gear permitted adjustment to be made at any time, but particularly when the work of joining up the projecting cantilever arms in the middle of the span had to be undertaken.

The longitudinal girders were built parallel to the old girders as shown in the section on page 120, but at a slightly higher level, while traffic was passing to and fro on the bridge. When completed, the new girders were lowered to the finished level by hydraulic jacks, and the cross girders for carrying the roadway were put in, partly in trenches across the old roadway and partly from below the deck of the old bridge. The new floor was then finished in sections; it is of buckle plating covered with concrete, on which wood paving is laid.

When the spans were thus completed, the old girders were removed piece by piece, and the new structure was moved laterally, again by hydraulic power, into its correct position on the cross girders on the piers.

There is a footpath outside of the main girder on each side, and the gas and water mains are carried at the extreme ends of the cross girders. The weight of steel in the superstructure is 2750 tons, and the cost of the whole work was about £82,000.

The North Bridge, Edinburgh.[1]

IN 1895 the firm reconstructed the North Bridge, which connects the old town of Edinburgh with Princes Street, a thoroughfare rightly regarded as the most picturesque in the United Kingdom. The bridge spans the valley at a point where this depression is occupied by the Waverley Station of the North British Railway; the view on page 125 shows the station in course of reconstruction, with the bridge beyond, and the historic castle in the distance.

The bridge, which was designed by Messrs. Blyth and Westland, of Edinburgh, differs from those which we have described, as its main feature is three segmental arches of 175 ft. span, springing from piers 18 ft. wide. The general effect is graceful, and worthy of the surrounding architectural and scenic features. Each span consists of six arched girders springing from abutments at each end, or from intermediate piers. The headway above rail level under the bridge is 28 ft. 3 in. at the springers at the north abutment, and 49 ft. 8 in. at the south abutment; the roadway is from 78 ft. to 79 ft. above rail level.

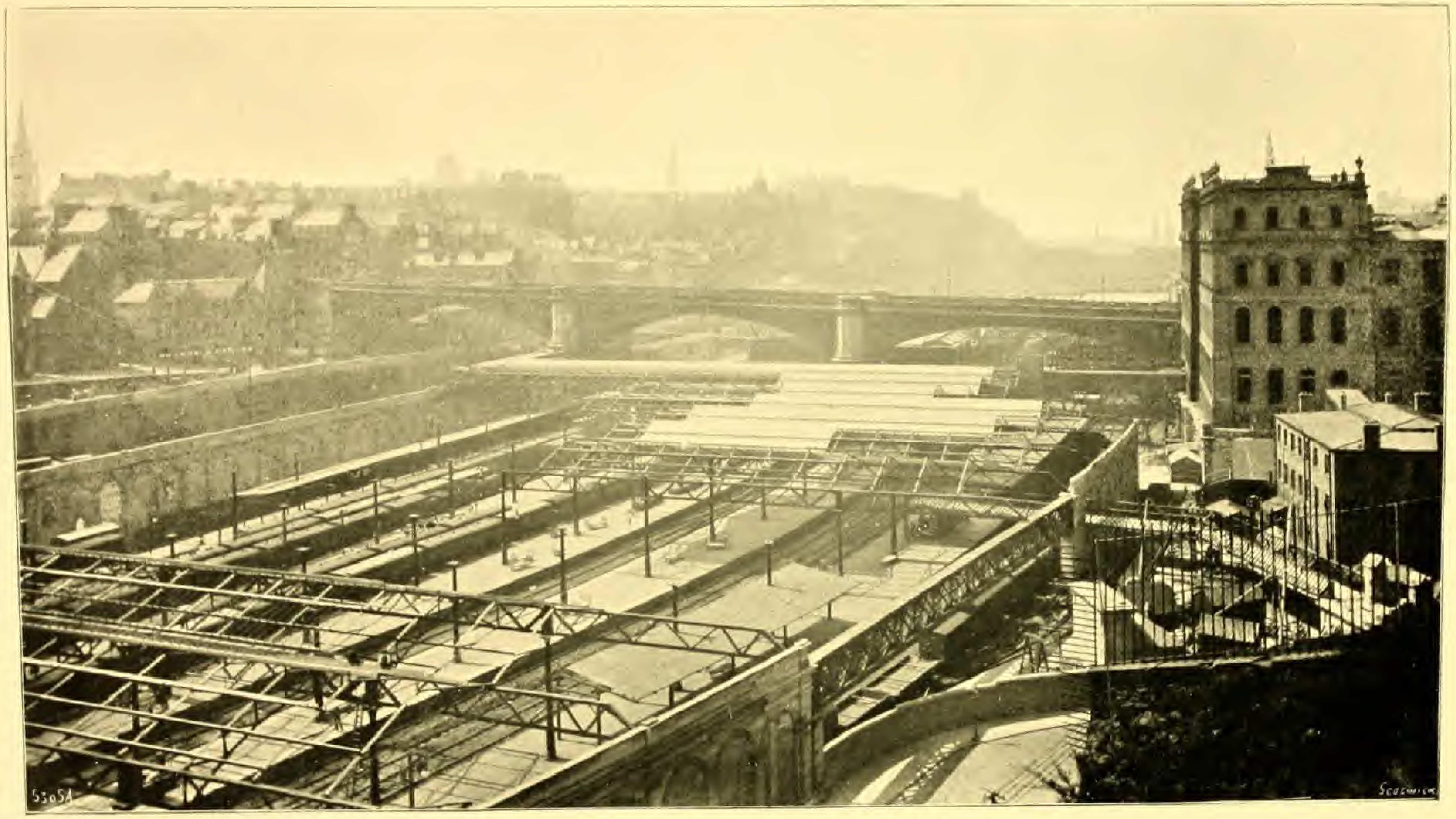
The piers, which are built of solid masonry, are 96 ft. 6 in. by 18 ft., with octagonal ends. The abutments are 82 ft. by 33 ft., and are square on the springing face. The cast-iron springers are of massive construction, and receive heavy cast-iron rockers, upon which the arched

[1] ENGINEERING, vol. lxviii., pages 423, 491.

ribs abut. These castings are 18 ft. long and 4 ft. $7\frac{1}{2}$ in. in height, and each weighs 13 tons. They pass through the pier from side to side. The six arched web girders in each span are 4 ft. deep, and have a rise of 22 ft. $1\frac{13}{16}$ in.; they are set at a radius to the soffit of 185 ft. 6 in.

From the extrados there runs parallel with the roadway a web girder 2 ft. 6 in. deep, which is also continued in a vertical line down the face of the pier to connect with the curved girder at the springing level; the spandril thus formed has lattice bracing. The bracing between the top horizontal and the bottom curved member is divided for the most part into 6 ft. bays. A roadway 75 ft. wide is carried on the top of this web girder. The footpath on each side is 11 ft. wide, leaving 53 ft. for the roadway, along which there is laid a tramway.

The cost of the work was £90,000. This included the removal of the old masonry bridge, which was at a lower level, and had a greater number of piers; the three main arches being only 70 ft. span, with several small side arches. There had therefore been considerable interference with the traffic.

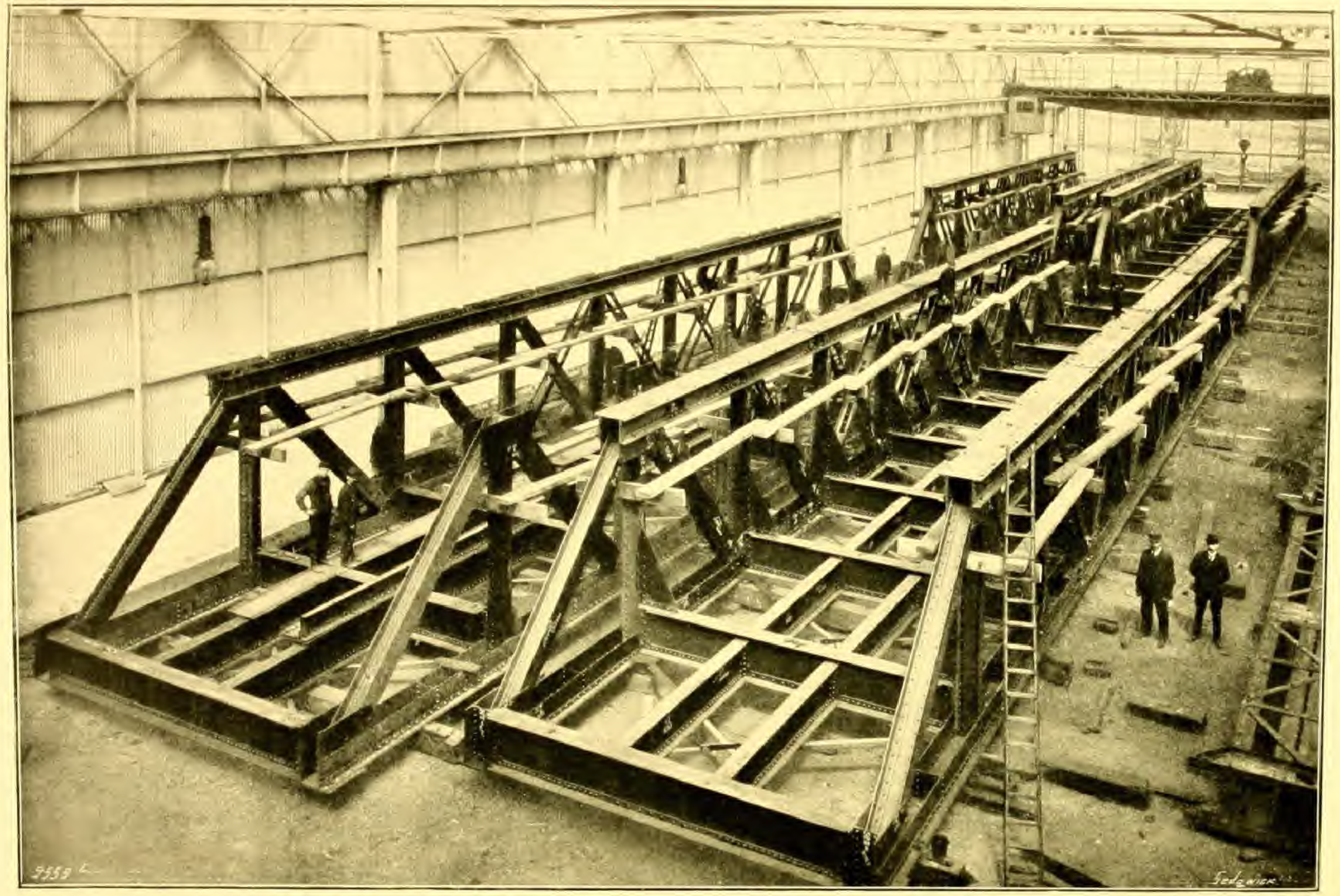


The North Bridge, Edinburgh, with Waverley Station under Reconstruction.

Sudan Railway Bridges.

THE illustration opposite shows a number of girder spans, designed and constructed at the works in Glasgow for export to the Sudan. Sir William Arrol and Company have supplied practically all the bridge-work for the railway from the new port of Suakin, on the Red Sea, to Khartûm, which owes its construction to the genius and enterprise of Lord Cromer. The bridges, which are constructed for the most part of N girders in standard lengths, the spans being 105 ft. and 55 ft., are used mostly for crossing rivers, the position of the piers and abutments being determined to suit the spans.

The railway has a total length of 332 miles, and will have an important influence in the development of the undoubtedly rich agricultural area of the Sudan, as it shortens the line of communication to the sea by 900 miles. After leaving Suakin it trends northwards, and then ascends to the plateau, some 3000 ft. above sea level. Running south-west across the desert, a waterless stretch of 50 miles, it reaches the Atbara River at a point about 20 miles above the junction of that waterway with the Nile. It joins the old Khartûm-Wady-Halfa military railway about a mile north of the Atbara Bridge, and thence the military line is used to Khartûm. It is intended to construct additional lines to develop the resources of the southern Sudan.

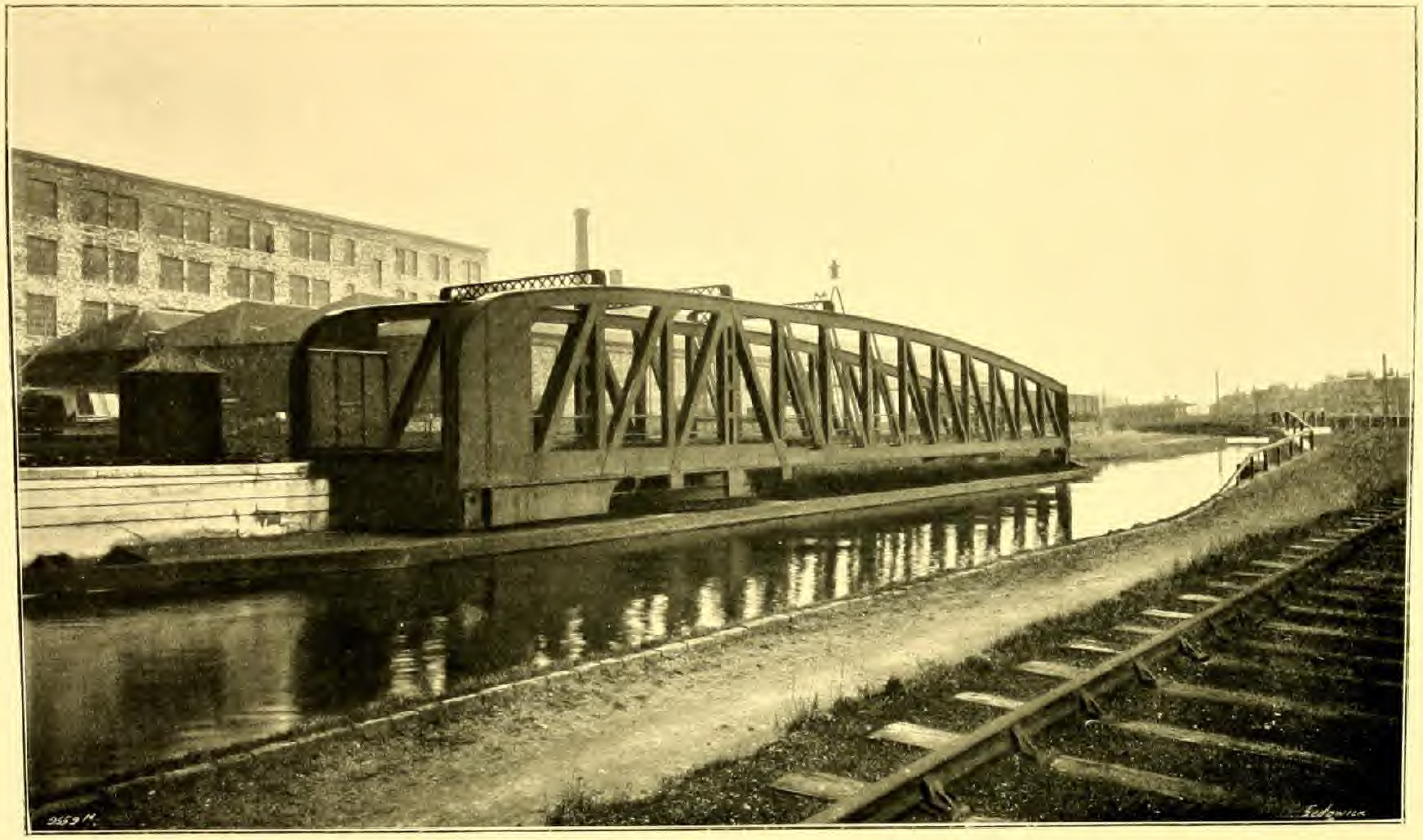


Sudan Railway Bridges: Girder Spans completed at the Glasgow Works for Ready Transport.

Swing Bridge for Railway Over a Canal.

ON the opposite page there is an engraving which illustrates a bridge designed, as well as built, by Sir William Arrol and Company, Limited, to carry the Lanarkshire and Dumbartonshire Railway over the Forth and Clyde Canal at Kilbowie. The waterway of the Canal is 22 ft. 3 in. in width, but it had to be crossed at an angle of 24 deg.; and the line was, moreover, curved in opposite directions on entering and leaving the bridge. This necessitated a long bridge, but notwithstanding this and the work involved in the turning gear, the bridge was completed in five months from the date of the order, at a cost of only £6600. The main girders, 17 ft. 6 in. deep, are respectively 152 ft. 7 in. and 133 ft. 3 in. long. The back end extends 36 ft. 4 in. from the centre of the pivot.

The pivot girder, and those adjoining it, together with the main girder on the top of the rollers, are of box section, shaped to suit their position, and of specially heavy scantlings. The cross-girders at the back end are of deep section, as shown in the view, and carry 180 tons of cast-iron kentledge blocks packed beneath the floor plates. The roller path and ring of rollers are 19 ft. 4 in. in diameter in the centre line. The bridge, which weighs 416 tons, is swung by hand-gear worked from a small platform on the side of the main girders, and can be opened in this way in fifty-three seconds or closed in sixty-eight seconds.



Swing Bridge for Railway Over a Canal.

The Dalginross Bridge at Comrie.

THE Dalginross Bridge over the River Earn at Comrie, shown on the engraving opposite was built in 1904, to replace an old masonry bridge with steep approaches and of insufficient width to allow two vehicles to pass each other. The distance between the abutments of the old bridge was 200 ft., and the river piers occupied 25 per cent. of the waterway, causing serious obstruction during floods.

The Local Authorities decided in 1904 to rebuild the bridge, and invited firms to submit proposals for its reconstruction. The new bridge was to have a width of roadway of 20 ft. between the kerbs, with a footway 5 ft. wide on each side, and a gradient not steeper than 1 in 30. The maximum constructional depth between the surface of the road and the flood-water level at the abutments was not to exceed $3\frac{3}{4}$ ft. As regards load, ordinary road traffic was to be provided for, in addition to a traction engine drawing two $8\frac{1}{2}$-ton wagons.

The design submitted by Sir William Arrol and Company, Limited, was accepted by the Committee as the one which complied with the stringent conditions in an economical manner, and was of a pleasing appearance. The new bridge is a deck structure, with no obstruction to a clear view of the surrounding country. The length of 200 ft. between the abutments is divided into two side spans of 55 ft. and a centre span of 90 ft., and the new piers occupy only 5 per cent. of the waterway.

The Dalginross Bridge at Comrie.

The bridge is a novel type of construction, and is the first of its kind constructed in this country. It is of the design known as "Constrained Cantilevers," in which the cantilevers are continued over the piers and meet in the centre of the span, where they are connected by a pendulum link. This type was adopted for æsthetic reasons, and on account of the small constructional depth allowed. The bridge has proved rigid in its construction, and agreeably free from vibration. Movements due to temperature are provided for at the abutments and in the centre of the middle span.

The river piers were founded by caissons $25\frac{1}{2}$ ft. long, 6 ft. wide, with semicircular ends, and 10 ft. deep. The upper portions of the piers and the abutments are built of sandstone. The bridge is carried on four main girders, at 6 ft. 4 in. centres, placed under the roadway. They are 18 in. deep at the abutments, $4\frac{1}{2}$ ft. at the piers, and 16 in. at the centre of the middle span. The roadways and footpaths are formed of tar concrete, about 6 in. thick over the crown of buckled plates. The footpaths are carried on cantilever brackets fixed on the outer main girders, and have an ornamental wrought steel hand-rail.

During the construction of the bridge, the road traffic was provided for by a temporary bridge 153 ft. long and 13 ft. wide.

The Manchester Ship Canal Bridges.

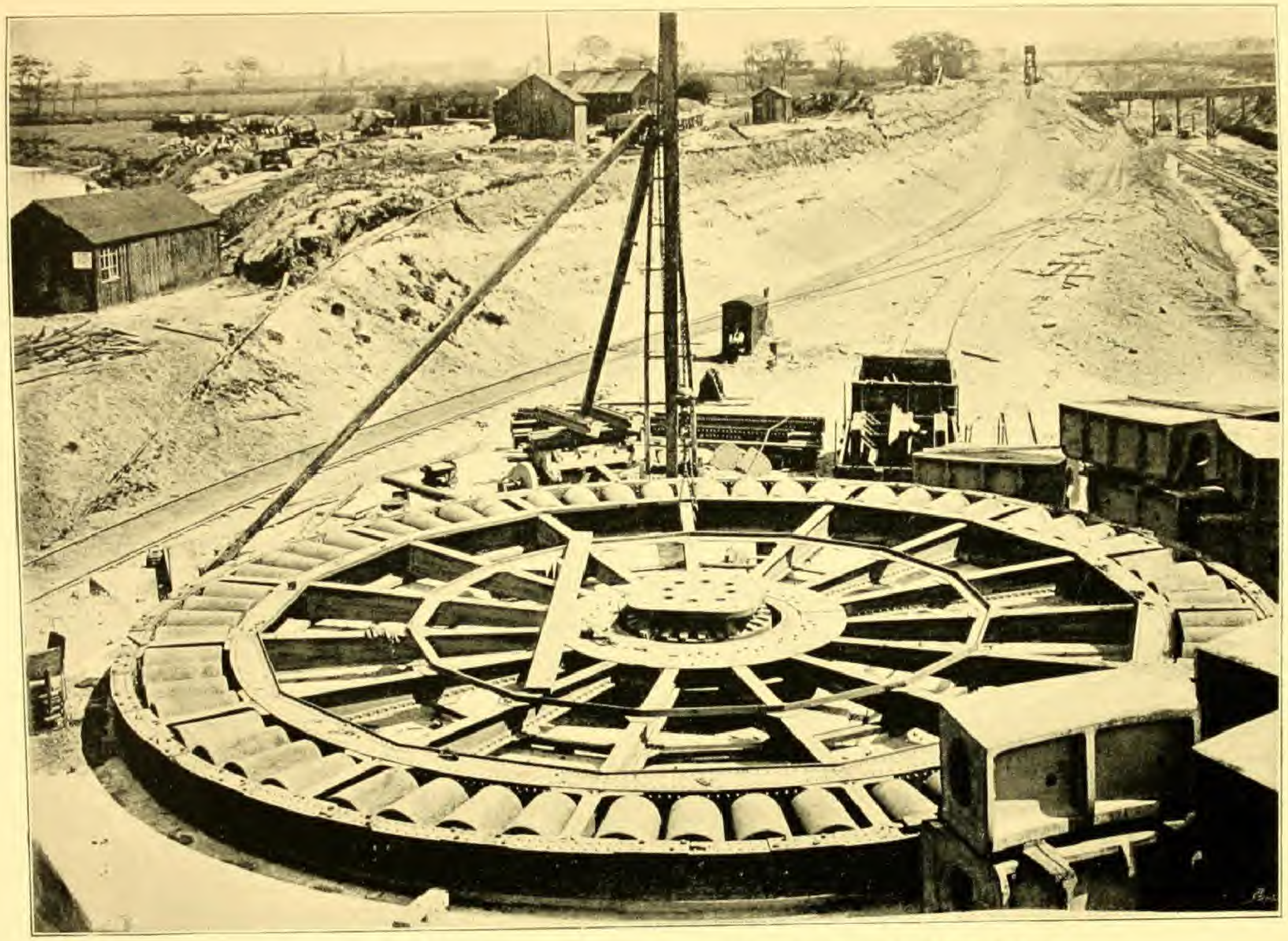
THE Manchester Ship Canal,[1] of a length of 35½ miles, necessitated the construction of a large number of bridges for the accommodation of highways and railways. In the case of railways where the traffic was extensive, the lines were deviated; so that, while the new bridges were at a height to give the desired clear headway of 75 ft. above the water surface, convenient approach gradients could be arranged. For roadways, and also for railways with a moderate volume of traffic, swing bridges were constructed.

It was natural that a firm of the experience of Sir William Arrol and Company, Limited, should take a large part in this bridge-building work, and of the eight large swing bridges, six were constructed by the firm; while of the heavier railway bridges, six also were built by them. As typical of the latter, we illustrate on the next page one of the principal railway structures, carrying the main line of the London and North-Western Railway. This bridge, it will be seen, was built on a heavy skew. The girders over the canal are of 298 ft. in length, and 36 ft. in depth at the centre, tapering to 24 ft. at the ends.

The opening bridges were built to turn on a roller path on one bank, so as to leave the canal without obstruction. The roller ring of one of the bridges is illustrated on page 135.

[1] See ENGINEERING, vol. lvii., page 97.

Bridge over Manchester Ship Canal for Railway.



Roller Ring for Stockton Heath Swing Bridge, Manchester Ship Canal.

The accompanying Table shows the principal dimensions of the swing bridges built :—

Name of Bridge.	Span.	Width of Road.	Length of Long Arm.		Length of Short Arm.		Depth of Main Girders at Centre over Angles.		Number of Rollers.	Diameter of Roller Circle.		Weight.
	ft.	ft.	ft.	in.	ft.	in.	ft.	in.		ft.	in.	tons
Old Quay, Runcorn	120	20	139	$\frac{9}{16}$	96	$10\frac{9}{16}$	28	0	60	22	11	650
Moore Lane	120	25	140	0	98	0	27	$8\frac{1}{2}$	64	27	$10\frac{1}{2}$	790
Stagg Inn	120	25	140	0	98	0	27	8	64	27	$10\frac{1}{2}$	790
Northwich Road ...	120	36	148	0	100	0	30	0	60	38	9	1350
Knutsford Road ...	120	36	148	0	100	0	30	0	60	38	9	1350
Barton Bridge ...	90	25	111	0	81	0	26	0	64	27	$10\frac{1}{2}$	640

The point of interest in connection with these various swing bridges is the method of turning them. Secured to the main structure through four cross girders is an annular girder, which, in the case of the heaviest bridge, was built up of 18 cast-iron segments, the upper flange being 3 ft. 3 in. wide, and the lower 2 ft. 10 in., strengthened by diaphragm plates. This annular girder has an outside radius of 15 ft. 5 in., and was built in concentric webs, each 1 ft. 10 in. in depth. A series of radial beams connects the annular girder with the centre bearing resting on the pivot on which the bridge rotates. The annular girder rests upon rollers, which again bear upon a fixed circular track of similar construction to the annular girder. As shown in the illustration, there are sixty rollers, conical in form to suit the radii. A bolt passing through each roller forms an axle, and is connected with a live ring on both sides; gun-metal washers are placed between the boss of the roller and the web of the live ring.

Blackfriars Bridge, London.

THE Blackfriars Bridge across the Thames[1] has a length between abutments of 922 ft., and is divided into five spans, varying from 155 ft. to 185 ft. Each span is formed of arched ribs, spaced about 10 ft. centres, supporting the roadway girders above the level of the crown of the arch.

In 1906 it was decided to proceed with the widening of the bridge from 75 ft. to 105 ft. between the parapets, to enable a double line of tramway to be laid across the bridge to connect the lines on the south side of the river with those on the Embankment. The contract for this work was let in 1907 to Sir William Arrol and Company, Limited, who undertook to complete it within three years.

The operation of widening the bridge involved the lengthening of the abutments and piers, with their foundations, in the line of the river, the transfer of the existing face ribs on the west side, with their cast-iron spandrils and hand-rail, along the extended piers 30 ft. further west, and the building of three new ribs between the older portion of the bridge and the outer rib in its new position.

In all the spans, except that next the north abutment, a clear passage for navigation had to be maintained throughout the progress of the work.

Across the north span and extending westwards, a timber staging was built, which served as a platform for receiving, handling, and storing material, and the new ribs

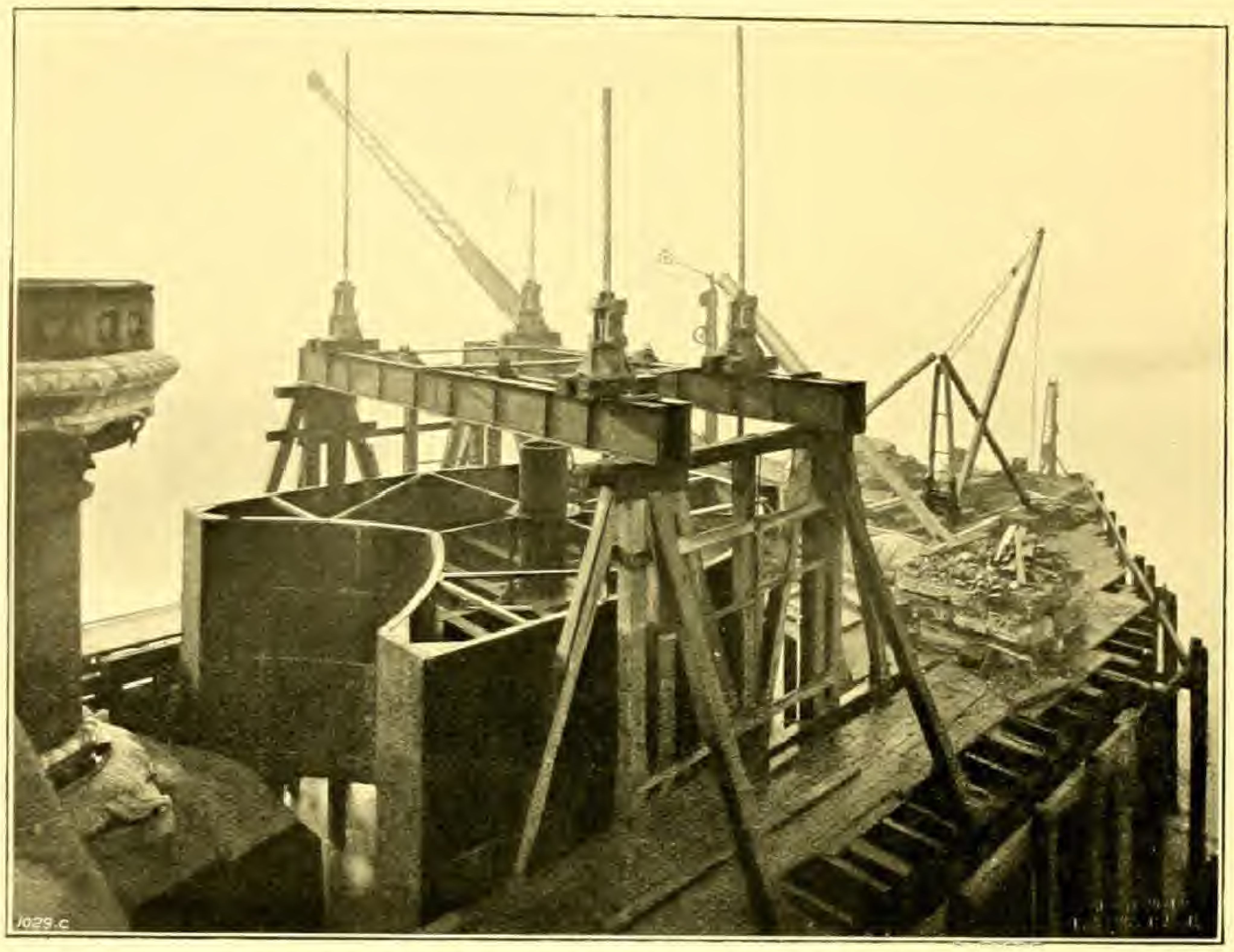
[1] See ENGINEERING, vol. lxxxiii., pages 75 and 853; vol. lxxxvi., page 251; vol. lxxxvii., page 310.

of this span were erected on trestles resting on the platform. In the case of the remaining spans a timber staging was built round the site of each of the new foundations, as shown in one of the views facing this page. At the south abutment a somewhat similar staging was placed round the cofferdam used there for foundation work.

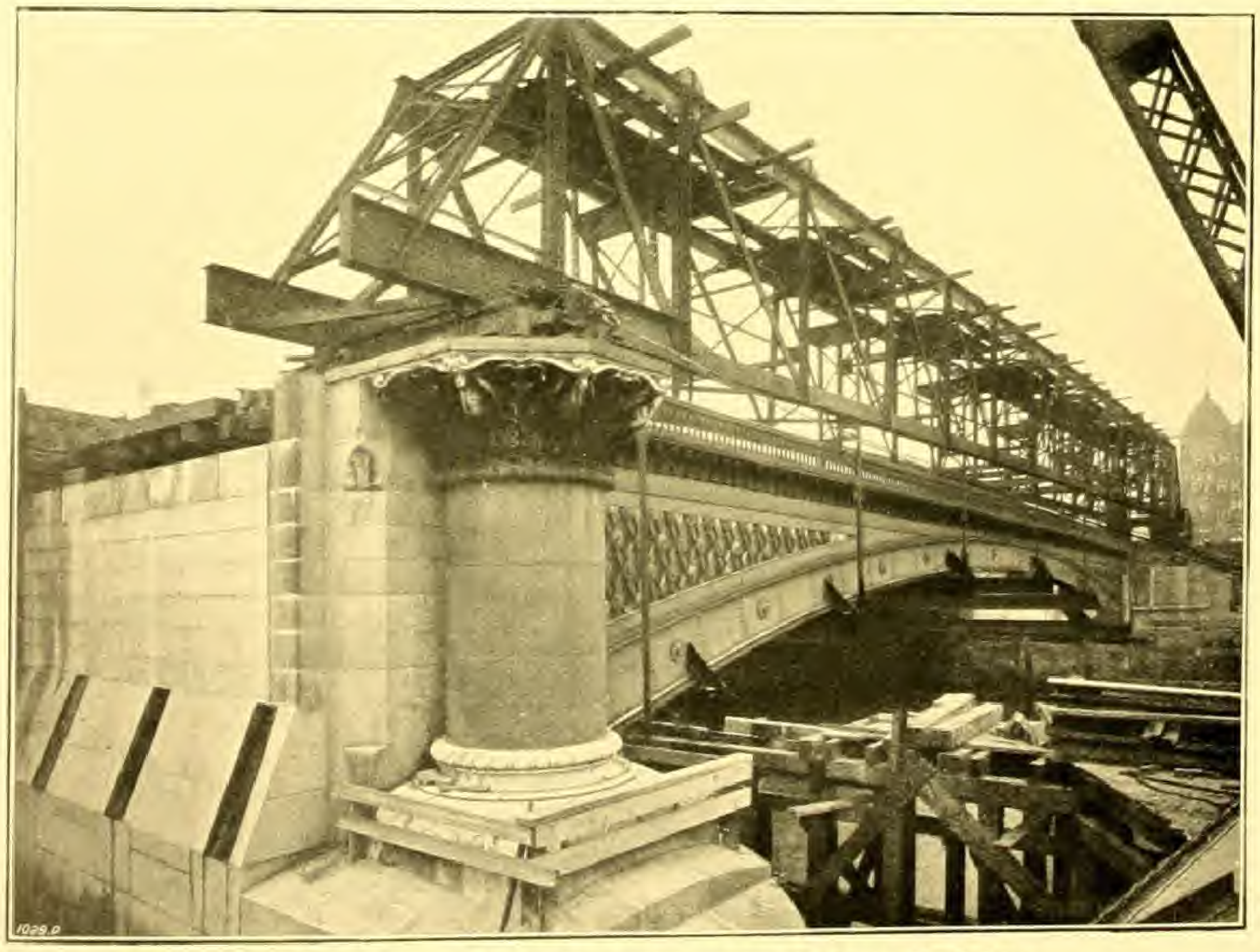
The sinking of the caissons at the north abutment was attended with considerable risk, owing to the adjacence of the Metropolitan District Railway tunnel, the old abutments, and the Embankment walls. The new caissons were carried considerably lower than the adjoining old foundations. When the three caissons forming the new abutment foundations were sunk and filled with concrete they were loaded with about 3000 tons of rails to consolidate the foundations and obviate any settlement after the bridge is completed. All the operations at the abutment were entirely successful, and were carried out without disturbance to any of the old works.

The foundations of the new part of each pier consisted of a caisson, shaped in plan with a cut-water point, and with a recess to fit the point of the existing caisson, as shown in the upper view on the opposite page. The caissons were built over their final position, and sunk by the pneumatic process in a manner similar to that adopted at the Clyde and Wear Bridges already described.

The erection of the steelwork in the north span presented no difficulty, as it was built *in situ* on the temporary staging. No scaffolding was permitted in the other spans, and the work of removing the existing face ribs was carried out by means of an overhead traveller, as shown in the engravings on the three following pages. It moved on rails laid on the top of the new portion of each pier at right angles to the centre line of the



Caisson for Pier Ready to be Lowered into Position.



Launching one of the Face Ribs into its New Position.

General View of Works, looking South-East.

bridge. From the overhead traveller there were suspended hangers, each of which was provided with a screw at the top, while the lower end was secured to the rib about to be shifted.

Rib just Launched, showing Temporary Girders Used for Steadying Rib during Launching.

When these preparations for lifting were completed, the floor between the face rib and the adjoining ordinary rib was removed. Cross-staging girders had previously been put in place and fixed to the face rib at one end. These girders were meantime held up below the old portion of the bridge, and passed through a special

stirrup bearing attached to one of the old ribs. The rib was raised from its bearings at the skewbacks by tightening up the screws on the top of the hangers. This operation was performed with extreme care, to ensure that the weight would be distributed equally upon the several hangers attached to the rib. When the rib was clear of the skewbacks, the overhead traveller was moved forward 30 ft. in short stages by means of a tackle operated from cranes on the staging. To prevent the rib swinging on the hangers, it was controlled by means of screws secured on the cross-staging girders between the old ribs. When the face rib was over its new position it was lowered to its correct level, and white or other metal run in between the abutting faces at the piers. The cross-staging girders were securely fixed to the face rib and the next old rib, spanning the distance between them and forming supports upon which to erect the three new ribs. The floor girders and cross frames were afterwards built. The heaviest rib moved weighed 150 tons, and the traveller 90 tons.

The face ribs of the respective spans were moved to the new position in the order which would produce least unbalanced thrust on the new portions of the piers, and the building of other portions of the superstructure was strictly limited to certain points for similar reasons.

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View of the Walney Viaduct from Vickerstown.

Viaduct over Walney Channel at Barrow-in-Furness.

WITHIN recent years, the Isle of Walney, which lies seaward of the town of Barrow-in-Furness, and is separated from it by a channel about a quarter of a mile wide, has become a residential district. The consequent growth of traffic across the channel has necessitated the superseding of the ferry steamer by a viaduct[1] designed by Messrs. Baker and Hurtzig, Westminster.

The total length of the viaduct between abutments is 1125 ft. There are eight fixed spans varying from 83 ft. to 118 ft. in length, and one opening span of the rolling-lift type, which gives a clear passage of 120 ft. for navigation. The roadway, on which there is a double line of tramway, is 31 ft. 4 in. wide, and the footpaths 9 ft. 4 in. All the piers are formed of pairs of cylinders, partly of steel and partly of cast iron, filled with concrete and bound together by a capsill girder at the top.

Each fixed span consists of two main girders of lattice construction, placed 31 ft. apart. They are connected by the roadway cross girders, which are riveted to the top of the vertical posts. Underneath these cross members the main girders are tied together by suitable diagonal bracing. The longitudinal stringers are placed over the cross girders, and support jack arches of concrete on which

[1] See ENGINEERING, vol. lxxxvi., pages 65, 69, 172, and 231.

the wood block roadway floor is laid. The footways rest on cantilever brackets riveted to the outside of the main girders.

The opening span is of the Scherzer type, with two leaves of plate-web construction, and with a clear opening of 120 ft. It is the largest span of this type in this country. Several improvements on the earlier rolling-lift bridges are embodied in it. In previous bridges the segments of the opening leaves were rolled along the fixed track girders by means of horizontal struts operated by machinery placed on the adjoining fixed spans. One end of each strut was fixed to a connection at the centre of the rolling segment, and the other end was geared to the machinery. In the Walney Bridge these struts are dispensed with, and the machinery placed direct on the opening span leaves. Alongside the track girders there are frames supporting fixed racks, into each of which is geared a large pinion wheel, keyed to a shaft passing through the centre of the rolling segment, or "the centre of rotation." When these shafts are actuated by the operating machinery, placed between the main girders of each leaf, the span is opened or closed. Electric power is used throughout, but hand gear, which may be operated from the permanent dolphin, is also provided.

All the opening and signalling operations are controlled from a cabin on the Walney side of the opening span. The controller levers are so interlocked that all signalling and opening operations can only be effected in proper sequence. The time required to open the bridge is 1 minute 45 seconds, and to close it to river traffic, 2 minutes 7 seconds, including all operations. Two 25 horse-power motors are provided for opening each leaf.

Permanent timber dolphins were constructed at each

View from Dolphin of Roller Lift Span Open for Steamship Traffic.

side to protect the piers and girderwork of the opening span from damage by shipping.

On the Barrow side of the bridge the abutments rest on clay, which was sufficiently stiff to enable excavations to be carried out with light timbering. Loose running sand was encountered on the site of the Walney abutment and necessitated the adoption of a full tide cofferdam.

For the erection of the bridge a staging was built across the channel on the line of the bridge. It consisted of pile trestles spaced about 16 ft. apart, supporting the longitudinal timbers and bogie tracks on which a travelling crane was placed. The cylinders of the piers were lowered to the river bed from the staging and were sunk under pneumatic pressure to their ultimate depth. The lower rings of the cylinders are of steel, and the upper rings of cast iron. During sinking, the cast-iron rings were not subjected to air pressure; the inner ring of concrete always extended a foot above the roof of the working chamber. The lower end of the working shaft was connected to this roof of concrete, and to the upper end the usual air locks were attached in a manner similar to that described on page 18.

The main girders of the fixed spans were erected in their final position from the temporary staging. In order to keep a clear waterway there was no staging in the centre channel, and the opening span was erected in an upright position. When nearly completed, the leaves were lowered to the closed position by the hand gear, in order that the last lengths, with the connections between the leaves, might be inserted. At the same time the balancing of the leaves was adjusted and the electrical operating gear installed.

STRUCTURAL STEEL WORK.

The Design of Workshops.

THERE are still extant in this country a sufficient number and variety of examples of old engineering shops to establish the remarkable development that has taken place during the past fifteen or twenty years in the design of factory buildings. The old shops, with their masonry walls and timber-trussed roofs, afforded splendid protection from weather, but were as well—or as ill-lighted—to quote a proprietor of such a shop—"at midnight on the 21st of December as at mid-day on the 21st of June." Such a condition has long ago been recognised as inimical to efficient and economical manufacture. The demand to-day is for complete glazing, so that there may be the minimum loss of natural light, with adequate weather protection.

It is now many years since Sir William Arrol and Company, Limited, built their first workshop on those lines; and since then they have, as the following pages suggest, reconstructed many old works and erected many admirably planned new establishments.

The modern factory building may not be any more beautiful than the old structures when viewed externally; but the interior is, as a rule, pleasing to the eye. The light steel truss has taken the place of heavy timber work, giving an impression of sufficient strength combined with a general sense of lightness and fitness of design. The lattice columns and girders suggest that

the material has been used in a way that takes full advantage of its qualities, while the long-drawn bays bespeak familiarity with constructional steelwork and studied economy in design and arrangement.

The first object in design is, of course, to build a shop suited to the business. Nowadays this practically amounts to a steel structure, to all intents and purposes independent of its surrounding walls. The width and length of bays are settled by the size and amount of work; the height depends on the size of work and the use to which cranes will be put; and the form of roof is chosen with due regard to lighting, ventilation, &c. The walls—of brick, masonry, galvanised iron, or weather-boarding—serve as protection to the contents against weather and depredation. Brick work is now being preferred for the walls in preference to the corrugated iron of earlier buildings, because the maintenance charges are lower, and the temperature within is higher in winter and lower in summer.

One of the chief considerations to be taken into account in designing details of steel work of the present day structure has reference to crane-power. Shops for small work present but few difficulties. Stiffness to resist wind pressure can be easily arranged for, while provision for lighting, ventilation, shafting, and good floors are important but simple *desiderata*. With large shops, in which crane loads of from 50 to 150 tons have to be provided for, important problems have to be solved, which, as indicated in the Appendix, demand experience and care. Columns which have to support the roof and crane, and also the structure as a whole, must be stiff enough to resist all strains due to the use of cranes and shafting, in addition to those due to external agencies, such as

wind and snow. These points have been subjects of careful investigation by Sir William Arrol and Company, Limited. There are, for instance, the forces on the structure of a shop due to cranes; the horizontal stresses produced in a building by the sudden application of the brakes of a crane or cranes, carrying a heavy load; the effect of the cross-travel of the crab; the stresses when cranes are used for dragging things along the shop-floor; the stiffness of columns and girders to withstand stresses due to combinations of loads such as those involved by overhead and jib-cranes working in close proximity to each other; and the allowances necessary for expansion.

These, stated at random, are only a few of the considerations involved in the design of workshops. On the four succeeding pages there is a list of the principal workshops built by Sir William Arrol and Company, Limited, and following it are brief descriptions and illustrations of typical buildings. In the Appendix are standard specifications, with other data and formulæ, which will assist the reader in appreciating the high efficiency aimed at in the buildings designed and constructed by Sir William Arrol and Company, Limited.

TABLE II.—PRINCIPAL WORKSHOPS BUILT BY SIR WILLIAM ARROL AND COMPANY, LIMITED, GLASGOW.

Name of Company.	Purpose.	Date.	Length.	Breadth.	Height.	Weight of Crane Load.
			ft. in.	ft. in.	ft. in.	tons
Ailsa Shipbuilding Company, Limited, Troon ...	Engineers' shop ...	1904	150 0	60 0	49 0	One of 30
Ailsa Shipbuilding Company, Limited, Ayr ...	Engineers' shop ...	1907	100 0	49 7	45 0	One of 15
Sir William Arrol and Company, Limited, Dalmarnock Iron Works	Engineers' shop ...	1898	315 0	128 0	55 6	One of 40
	Girder shop ...	1905	775 0	162 0	42 0	Four of 10, and one of 20
	Gantry over yard ...	1905	230 0	122 0	25 0	One of 10
Babcock and Wilcox, Limited ...	Foundry ...	1895	200 0	122 0	43 0	One of 10
	Machine shops ...	1895	310 0	240 0	29 ft. to 36 ft.	20
	Do. ...	1900	350 0	216 0	25 ft. to 40 ft. and 13 ft. to 16 ft. 6 in.	One of 30 and 2 of 5
Barclay, Curle and Company, Limited, Glasgow	Erecting shop ...	1907	150 6	53 0	70 ft. 3 in.	—
Barr and Stroud, Glasgow ...	Workshops ...	1906	160 0	150 0	20 ft. 6 in. to 46 ft. 6 in.	—
William Beardmore and Company, Limited, Dalmuir	Engine and boiler shops ...	1903	720 0	323 0	75 0	Four of 60, one of 5, two of 30, one of 20, and one of 10
	Structure over ships' berth ...	1905	744 0	115 0	150 0	Two of 15 and 5-ton jibs.
Blackwood and Gordon (now Clyde Shipbuilding and Engineering Company, Limited), Port Glasgow	Engine shop ...	1899	100 0	39 0	45 0	Two of 30
	Boiler shop ...	1898	89 0	61 0	36 0	—
	Pattern shop ...	1903	120 0	60 0	39 0	—
Bow Generating Station (Charing Cross, City and West End Electric Company, Limited)	Boiler house ...	1901	300 0	77 0	93 0	—
	Engine house ...	1901	300 0	76 0	93 0	One of 30
British Electric Plant Company, Limited, Alloa	Machine shop ...	1901	332 0	120 0	33 0	Three of 10
	Erecting shop ...	1901	270 0	40 0	44 0	One of 15
	Engine house ...	1901	84 0	32 0	35 0	—
	Boiler house ...	1901	156 0	40 0	26 0	—
John Brown and Company, Limited, Clydebank (formerly Clydebank Shipbuilding and Engineering Company, Limited)	Funnel shop ...	1906	180 0	77 0	57 0	One of 15
	Erecting shop ...	1906	241 0	129 0	72 0	Four of 60
	Beambenders' shop ...	1899	150 0	90 0	41 0	—
	Machine shed ...	1901	360 0	180 0	44 0	4
	Shed over ironworkers' boards ...	1898	94 0	49 0	30 0	—
	Moulding loft ...	1896	376 0	53 0	38 0	—
	Sawmill ...	1898	200 0	132 0	52 0	Two of 5
	Shop for sheet-iron workers ...	1896	204 0	92 0	25 0	—
	Roof over scrieve boards ...	1897	182 0	54 0	33 0	—
John Brown and Company, Limited, Sheffield	Workshops ...	1905	262 0	256 0	63 0	One of 75, two of 30, two of 25, and two of 10
Carlisle Citadel Station ...	Roofs over station ...	1880	1020 0	283 0	—	—

Charing Cross and Strand Electric Lighting Company, Limited, Beech Street Sub-Station	... ...	1903	94 0	89 0	68 6	Two of 10 and one of 5
Coventry Ordnance Works, Limited, Scotstoun	Erecting shops	1906	675 0	134 0	63 0	Three of 10, two of 30, three of 10, two of 60, one of 100
	Outside gantry	1906	675 0	87 0	50 0	One of 100
David Rowan and Company, Limited, Glasgow	Engine fitting shop	1896	150 0	65 0	56 0	—
	Boiler shop	1898	234 0	150 0	56 0	Two of 40, and one of 25
	Do.	1906	189 0	175 6	65 0	One of 10, and one of 20
	Engine fitting shop	1906	220 0	57 0	57 0	One of 20
Dowrie and Company, Port Said	Engineers' shop	1907	180 0	75 0	50 0	One of 10
Dunsmuir and Jackson, Govan...	Boiler shop...	1905	304 0	135 0	42 6	Two of 60 and one of 15
Fairfield Shipbuilding and Engineering Company, Limited	Fitting shop (platers')	1897	132 0	130 0	31 6	—
	Brass foundry	1898	204 0	96 0	44 0	One of 30
	Moulding loft	1897	275 0	60 0	31 6	—
	Mechanics' shop	1901	200 0	130 0	28 0	One of 10
	Fitting shop	1898	291 0	55 0	73 0	One of 50
	Angle iron smithy and beam shed	1902	129 0	109 0	33 0	—
	Brass-finishers', coppersmiths' and galvanizing shops	1901	130 9	156 6	31 0	—
	Boiler shop...	1897	300 0	60 0	55 6	One of 20
	Joiners' shop	1897	240 0	160 0	37 0	—
	Store and workshop at dock	1906	160 0	25 0	20 0	—
Fenchurch Street Electric Sub-Station ...	... ...	1900	80 0	166 0	68 0	10
Thomas Firth and Sons, Limited, Sheffield ...	Heavy tool shop	1907	300 0	86 0	53 0	One of 70 and one of 40
Thomas Firth and Sons, Limited, Tinsley ...	Rolling mills	1907	595 0	240 0	36 0	One of 5
Glasgow Corporation Gas Provan Works ...	Coal stores	1902	405 0	296 0	50 0	—
Glasgow Corporation Tramways, Pinkston Station	Coal stores	1900	121 0	82 0	66 0	—
Glenfield and Kennedy, Limited, Kilmarnock	Iron foundry	1900	250 0	149 0	67 0	60-ton traveller and 7-ton jibs on columns
	Do.	1905	251 0	137 0	44 0	15
Great Northern Railway Goods Warehouse, Manchester	Stores	1897	265 0	218 0	71 0	—
A. Guinness, Son and Company, Limited, Dublin	Brewery storehouse	1903	165 0	146 0	116 0	—
Hall, Russell and Company, Aberdeen... ...	Boiler shop	1903	176 0	60 0	54 0	One of 50
Hawthorn, Leslie and Company, Limited, Newcastle-on-Tyne	Smithy	1904	123 0	62 0	37 0	—
	Iron store	1904	54 0	21 0	17 0	—
	Copper shop and brass foundry	1904	100 0	46 0	40 0	One of 10
	Store	1901	100 0	46 0	32 6	—
James Howden and Company	Engineers' shops	1897	150 0	170 0	33 0	—
	Foundry	1897	230 0	60 0	41 6	One of 10 and one of 5
	Pattern shop	1897	132 0	26 0	34 6	—
	Sheet-iron workers' shop	1899	102 6	92 6	31 0	—
Jonas and Colver, Limited, Sheffield	Steelworks	1906	266 0	316 6	45 6	One of 10, one of 20, one of 30
Lanarkshire Steel Company	Do.	1898	100 0	80 0	54 6	—

TABLE II.—PRINCIPAL WORKSHOPS BUILT BY SIR WILLIAM ARROL AND COMPANY, LIMITED, GLASGOW.—*Continued.*

Name of Company.	Purpose.	Date.	Length.	Breadth.	Height.	Weight of Crane Load.
			ft. in.	ft. in.	ft. in.	tons
London and Glasgow Engineering and Shipbuilding Company, Limited	Boiler shop...	1907	185 0	120 0	70 0	Four of 60
London Brothers, Johnstone	Machine shop	1898	147 0	46 0	46 6	One of 20
Marshall, Sons and Company, Limited, Gainsborough	Boiler shop...	1902	400 0	175 0	56 0	Two of 30, three of 20, four of 8, and two of 5
Martin and Millar and Millars, Limited, Glasgow (Tannery)	Lime shed	1886	210 0	144 0	25 6	
	Liquor department	1886	200 0	94 0	25 6	—
	Bark barn	1886	169 0	30 0	55 0	—
	Drying shed	1886	212 0	33 0	60 0	—
	Drying shed and store	1901	112 0	33 0	50 0	—
McFarlane, Lang and Company, London ...	Bakery	—	304 0	220 0	25 6	—
	Warehouse	1903	70 0	64 0	46 0	—
	Engine house	—	56 0	43 0	24 0	—
	Stables and shed	—	219 0	63 0	32 0	—
McKie and Baxter, Govan	Engineers' shop	—	160 0	57 0	33 0	One of 20
	Pattern shop and smithy...	1906	142 0	31 0	39 0	—
A. and P. McOnie, Govan	Engineers' shop	1895	170 0	82 0	55 0	One of 40
Metropolitan Electric Supply Company, Limited, Willesden	Boiler and engine houses	1900	390 0	189 0	62 0	—
Metropolitan Electric Supply Company, Limited (North Street)	Sub-station	1902	92 0	68 0	47 0	—
Metropolitan Electric Supply Company, Limited	Workshop	1904	90 0	50 0	33 0	One of 10
Neilson, Reid and Company, Limited, Glasgow (now North British Locomotive Co.)	Erecting shop	1897	706 0	44 0	54 0	One of 75, and two of 25
	Paint shop	1901	300 0	104 0	50 0	One of 10
	Cab shop	1903	215 0	28 0	49 0	One of 10
	Platers' shop	1900	210 0	176 0	45 0	Three of 10
North British Locomotive Company, Limited, Polmadie, Glasgow	Machine shops	1906	217 0	200 0	40 0	Three of 7 and one of 10
	Erecting shop	1906	332 0	120 0	56 0	Two of 90 and 2 of 30
	Tender shop	1906	332 0	70 0	53 0	Two of 40
	Joiners' shop	1906	332 0	50 0	43 0	One of 7
	Tinsmiths' shop and stores	1906	204 0	40 0	31 0	—
	Wheel shed	1906	210 0	40 0	42 0	One of 7
	Paint shop	1906	150 0	59 0	35 0	—
North British Railway Company, Limited ...	Leith Central Station	1902	810 0	220 0	70 0	—
Parsons Turbine Companies	Machine fitting and erecting shop	1899	385 0	80 0	44 0	Two of 40
	Extension of machine shop	1905	358 0	50 0	58 0	Two of 60
	Pattern shop	1899	150 0	40 0	30 0	—
	Brass foundry, copper and smiths' shops ...	1899	150 0	120 0	30 0	—
	Test house	1899	112 0	42 0	44 0	—
Ransomes and Rapier, Ipswich	Fitting shop	1898	116 0	44 0	53 0	One of 25

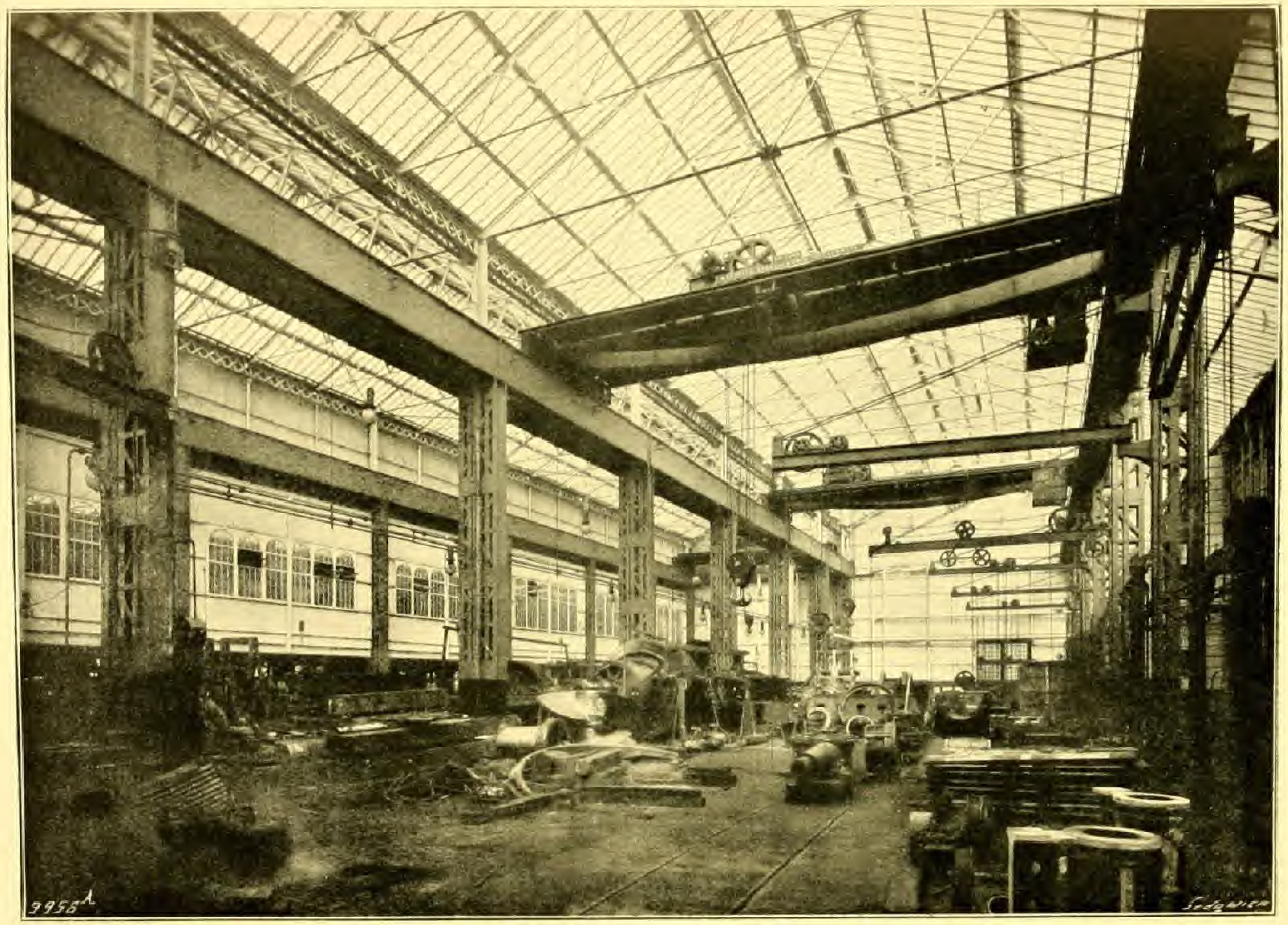
St. Martin's Lane Electric Sub-Station ...	... ...	1901	80 0	60 0	90 0	10 and 2
Scotts' Shipbuilding and Engineering Company, Limited, Greenock	Engineers' shop	& 1902	280 0	160 0	60 0	One of 40, one of 15, one of 5
	Boiler shop	1890	250 0	62 0	65 0	One of 100
Smith's Dock Company, N. Shields	Sheds	1899	180 0	80 0	32 0	—
	Do.	1900	120 0	120 0	38 0	—
John Spencer and Sons, Limited, Newburn Steel Works	Fitting shop	1898	335 0	77 0	51 0	One of 50
Steel Company of Scotland, Limited	Foundry	1899	214 0	70 0	68 0	Two of 60
	Shed over plate mills	1905	347 0	52 6	42 0	One of 30
	Dressing shed	—	188 6	52 0	—	One of 25
	Shop over soaking pits	1906	150 0	80 0	47 0	Two of 5
	Machine shop extension	1906	100 0	52 0	50 0	One of 25
	Roof over plate mill	1906	200 0	57 0	40 0	One of 15
	Roof over cogging mill	1907	130 0	57 0	40 0	Two of 15
Stewart and Lloyd's, Limited	Tube work (Imperial)	1906	600 0	480 0	36 0	7-ton jib cranes
	Engineers' shop	1898	200 0	112 0	40 0	One of 10
	Tube work (British)	1901	200 0	50 0	28 0	—
	Tool room	1905	84 0	40 0	35 0	—
James Templeton and Company, Glasgow ...	Carpet factory store	1897	110 0	87 0	75 0	—
Tranmere Bay Development Company, Birkenhead (Cammell Laird and Company, Limited)	Erecting shop	1906	1075 0	77 0	74 0	Two of 70
	Machine shops	1906	1135 0	42 6	50 0	35
	Smiths' shop	1906	270 0	42 6	30 0	15
	Light iron and pipe shop	1906	315 0	42 6	20 0	—
	Boiler shop	1906	500 0	130 0	69 0	Two of 70 and 1 of 15
Vickers Son and Maxim, Limited, Barrow-in-Furness	Iron foundry	1900	264 0	150 0	55 0	Two of 45, and one of 25
	Joiners' shops	1901	250 0	175 0	30 0	—
Wallsend Slipway and Engineering Company, Limited	Erecting shop	1905	570 0	60 0	65 0	Two of 60
	Smiths' shop	1897	160 0	56 0	32 0	—
	Pattern shop	1897	120 0	70 0	26 6	—
	Brass foundry, plumber and copper shop ...	1897	120 0	70 0	33 6	One of 10
	Boiler shop...	1904	330 0	75 0	70 0	One of 70, and one of 100
G. and J. Weir, Limited, Cathcart	Fitting shop	1902 & 1906	372 0	41 0	43 0	One of 30, and one of 5
	Iron foundry	1902	210 0	105 0	44 0	Two of 10, and one of 5
	Smithy and brass shop	1903	259 0	101 0	44 0	One of 15
Messrs. Williamson, Tanners, Canterbury ...	Drying shed	1894	120 0	30 6	34 7	—
Wylie and Lochhead, Limited, Glasgow ...	Furniture stores	1901	109 0	45 0	72 0	—
Yarrow and Company, London...	Boiler shop, platers' shop, and machine shops ...	1900	390 0	205 0	50 0	—
Yarrow and Company, Glasgow...	Boiler shop	1906	303 0	153 0	56 0	Two of 5, two of 10, one of 20, and one of 50
	Machine shop	1906	248 0	155 6	56 0	Three of 5, one of 20, one of 50
	Platers' shop	1906	182 0	92 0	31 0	—
	Pattern-makers' and joiners' shops	1906	375 0	47 0	39 0	—
	Blacksmiths' shop	1906	150 0	42 0	33 0	—
	Crane gantry over dock	1906	300 0	92 0	70 0	One of 50 and one of 10
	Galvanising shop	1907	90 0	30 0	20 0	—
	Crane gantry in yard	1907	330 0	85 0	26 0	Two of 7

John Brown and Co., Ltd., Clydebank.

THERE has been a long association between Sir William Arrol and Company, Limited, and the Clydebank Shipbuilding Yard,[1] one of the foremost naval construction establishments in this country. In that yard there has been built a long series of powerful warships and record-breaking Atlantic liners. The wealth of experience, alike in management and manufacturing methods, is indicated by the fact that the list of warships built, so far, terminates with the powerful battleship "Hindustan" and the greatest cruiser yet designed, the "Inflexible," which combines with an unexampled armament of eight 12-in. guns a speed of 25 knots; while the latest of the merchant vessels is the Cunard liner "Lusitania," of 32,500 tons, also to attain a speed of 25 knots.

Many of the buildings at the Clydebank Works have been designed and constructed by Sir William Arrol and Company, Limited. The shop most recently completed is the erecting shop, where the immense turbines for the new Cunard liners and the cruiser have been built. This shop is 241 ft. long and 129 ft. wide, with a height of 72 ft., and the structural details have been arranged to accommodate in each bay two cranes with a combined lifting capacity of 120 tons. In this shop the firm have undertaken probably more turbine machinery than any other firm, the total horse-power of such engines, finished

[1] See ENGINEERING, vol. lxxii., pages 242, 275.



Steam Turbine Erecting Shop at Clydebank Works.

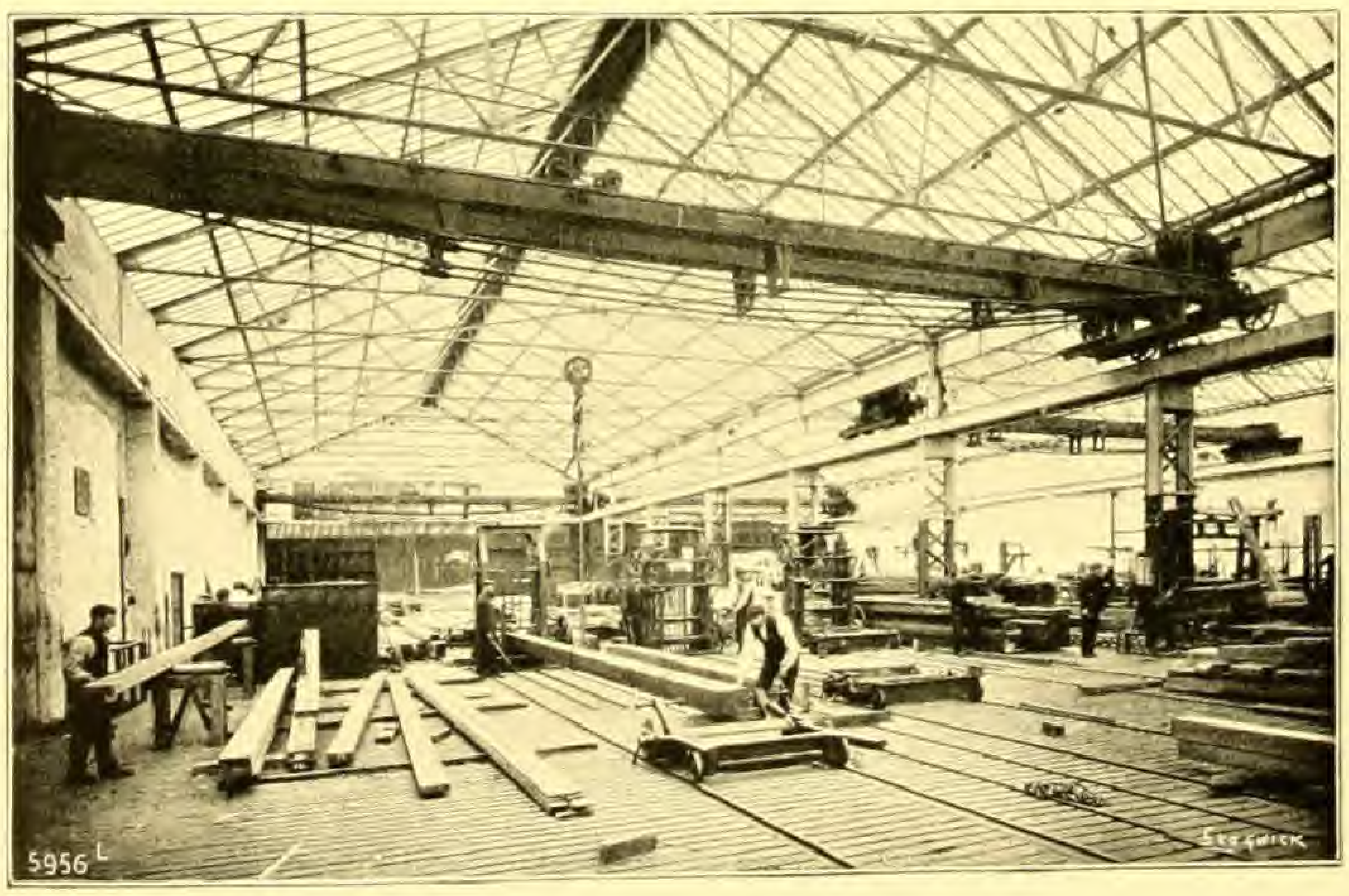
or in course of construction in January, 1907, being 129,000 horse-power.

Two other striking buildings are illustrated on the opposite page. The moulding loft, which is 376 ft. long and 53 ft. wide, is one of the finest buildings of the character yet completed, being particularly well lighted. On the floor of this building the lines of the ships to be built are drawn full size for the construction of the templates which are subsequently used for the machining of frames, beams, plates, etc., to be worked into the ships.

The other building illustrated is the saw-mill, 200 ft. long and 132 ft. wide, with a height of 52 ft. The engraving indicates the satisfactory character of the design.

At the Sheffield Works of the Company, Sir William Arrol and Company, Limited, have erected a workshop 262 ft. long and 256 ft. in width, with a height of 63 ft. The largest of the seven overhead cranes carried by the columns of the building lifts a load of 75 tons.

Moulding Loft at the Clydebank Works.



Sawmill at the Clydebank Works.

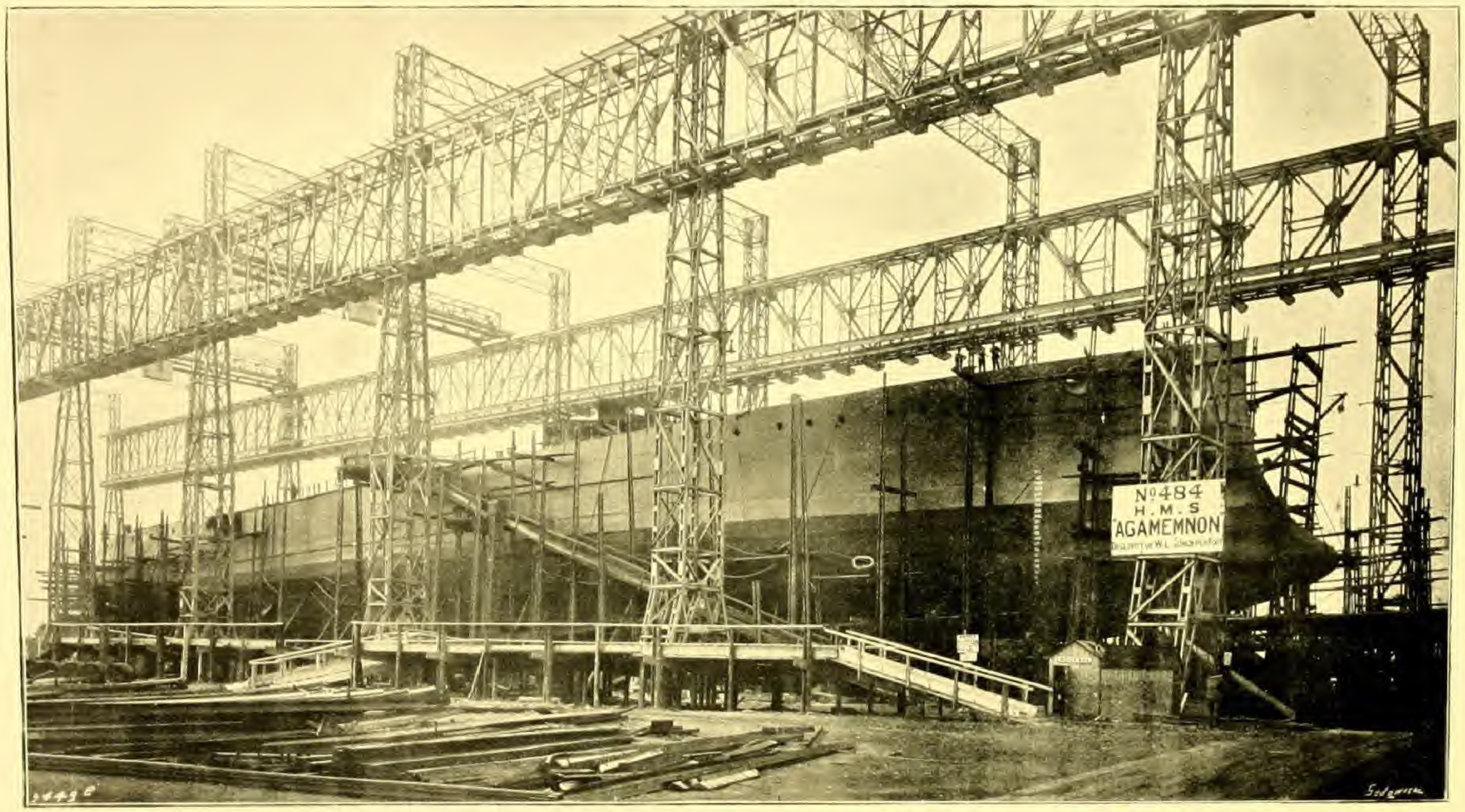
William Beardmore and Co., Ltd., Dalmuir.

ONE of the largest, and at the same time one of the finest, naval construction works in this country is the new establishment of Messrs. William Beardmore and Co., Ltd., at Dalmuir.[1] As those responsible for the planning of the establishment had, to use an historical phrase, a "clean slate," there was no hindrance to the realisation of the best possible scheme to meet modern conditions. There was, initially, a clear conception of the full extent of the requirements: it was intended that the best of naval and merchant work should be undertaken. At the same time it was decided that every approved system of modern manufacture should be adopted, with a view not only of dealing with the work in the most efficient manner, but also of ensuring that the highest degree of economy should be realised.

The new works have an area of about 90 acres, and a river frontage of nearly a mile—to be exact, 4920 ft. The shipbuilding berths have been arranged to take vessels up to 1000 ft. in length and over 100 ft. beam.

One of the most interesting problems in connection with the equipment of the shipbuilding yard was the arrangement of cranes for lifting material on board vessels in course of construction. Time was when ordinary shear-

[1] See ENGINEERING, vol. lxxviii., page 455.



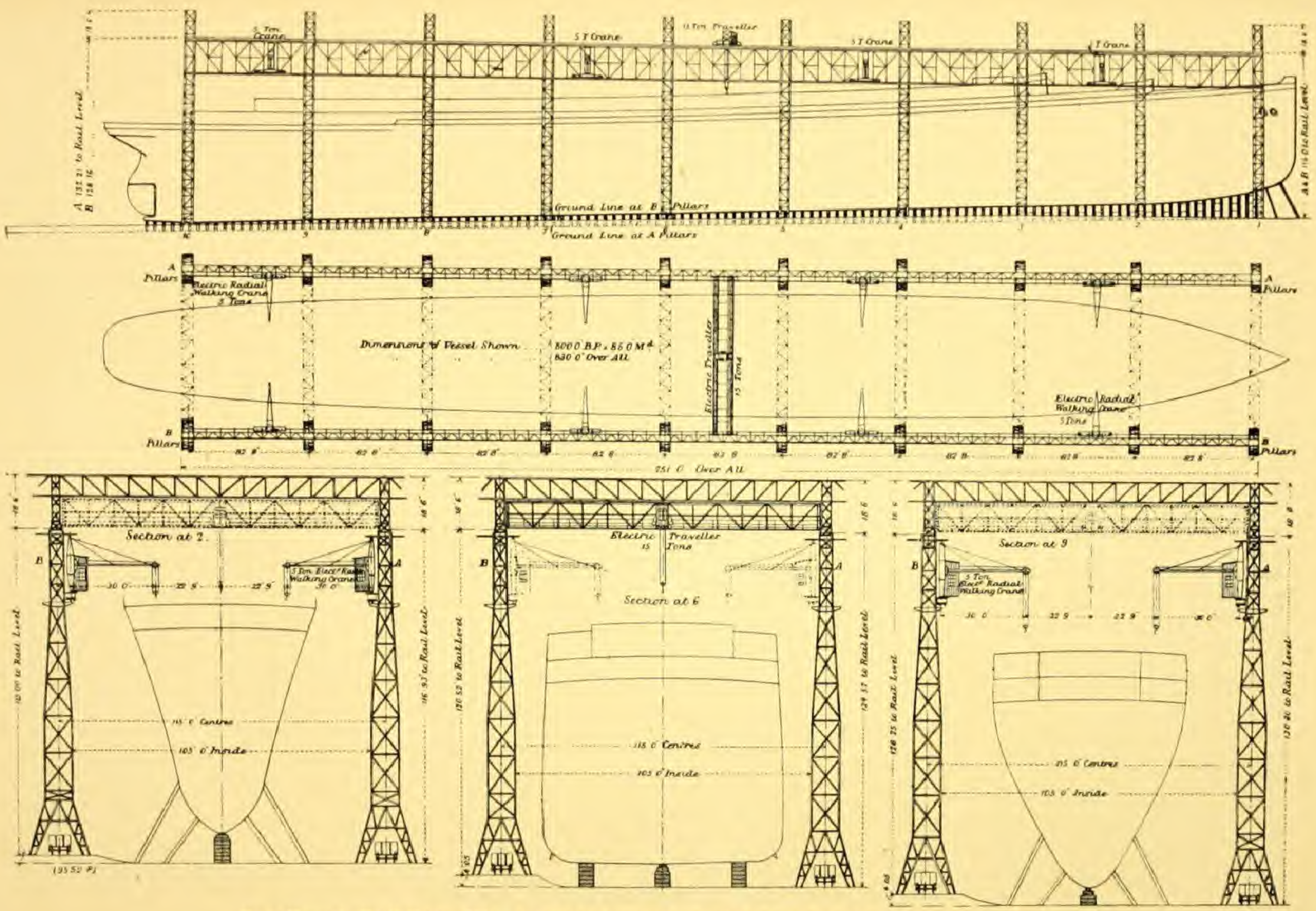
View of Building Berth, with His Majesty's Battle-ship "Agamemnon."

legs, and other temporary devices, were sufficient for most purposes; but with the great advance in the size of ships, and the increased weight of plates and angles worked into their structure, it has become necessary, in order to economise time, to provide more perfect crane appliances, for the crux of success in naval and merchant shipbuilding now is rapid construction.

The arrangement of the principal berth for the construction of battleships is illustrated by the longitudinal plan and cross-sections on the opposite page, and by an engraving on page 151, which shows also His Majesty's battleship "Agamemnon," the largest battleship yet built on the Clyde. The berth structure was designed and erected by Sir William Arrol and Company, Limited.

It will be noted that there are on each side of the berth four jib travelling or walking cranes, capable of lifting 5 tons, and having an overhead reach of 30 ft. Each of these cranes can travel the full length of the berth on rails supported by the vertical members of the building, and they may be congregated, if necessary, to deal with exceptionally heavy loads. At the same time there is a high-speed travelling crane stretching from one side to the other of the building berth, and capable of lifting 15 tons. This crane is specially useful for lifting weights into the centre line of the ship, while the jib cranes are at the disposal of the several plater or fitter squads engaged on the bottom or side shell plating.

Eight squads can be at work on a ship, each having a crane for dealing with material. The angle and plate trucks pass down each side of the berth, between the legs of the vertical members of the structure, as shown in the cross-sections, so that there is the minimum of obstruction; the empty wagons return on an outside track.



Details of Building Slip at Beardmore's Naval Construction Works, Dalmuir.

The superstructure over the berth is 750 ft. long, 135 ft. wide, and 150 ft. high at the end nearest the river. The roofing is not glazed. The vertical members of the steel work are carried on concrete foundations. Heavy timber piling has been put in throughout the length of the berths, with special rows, having suitable caps, under the keel blocks, under the position occupied

William Beardmore and Company's Boiler Works.

by the ways, and under the bilge blocks. The piling is very close near to the water's edge, where there will be the maximum thrust when the stern of the vessel first floats in the process of launching.

The boiler and engine shops were also designed and constructed by Sir William Arrol and Company, Limited, and of these engravings are published on this and opposite pages. The building, which includes both shops, is probably one of the finest yet constructed. The length is 720 ft., and the width 323 ft., in five bays, so that the total area

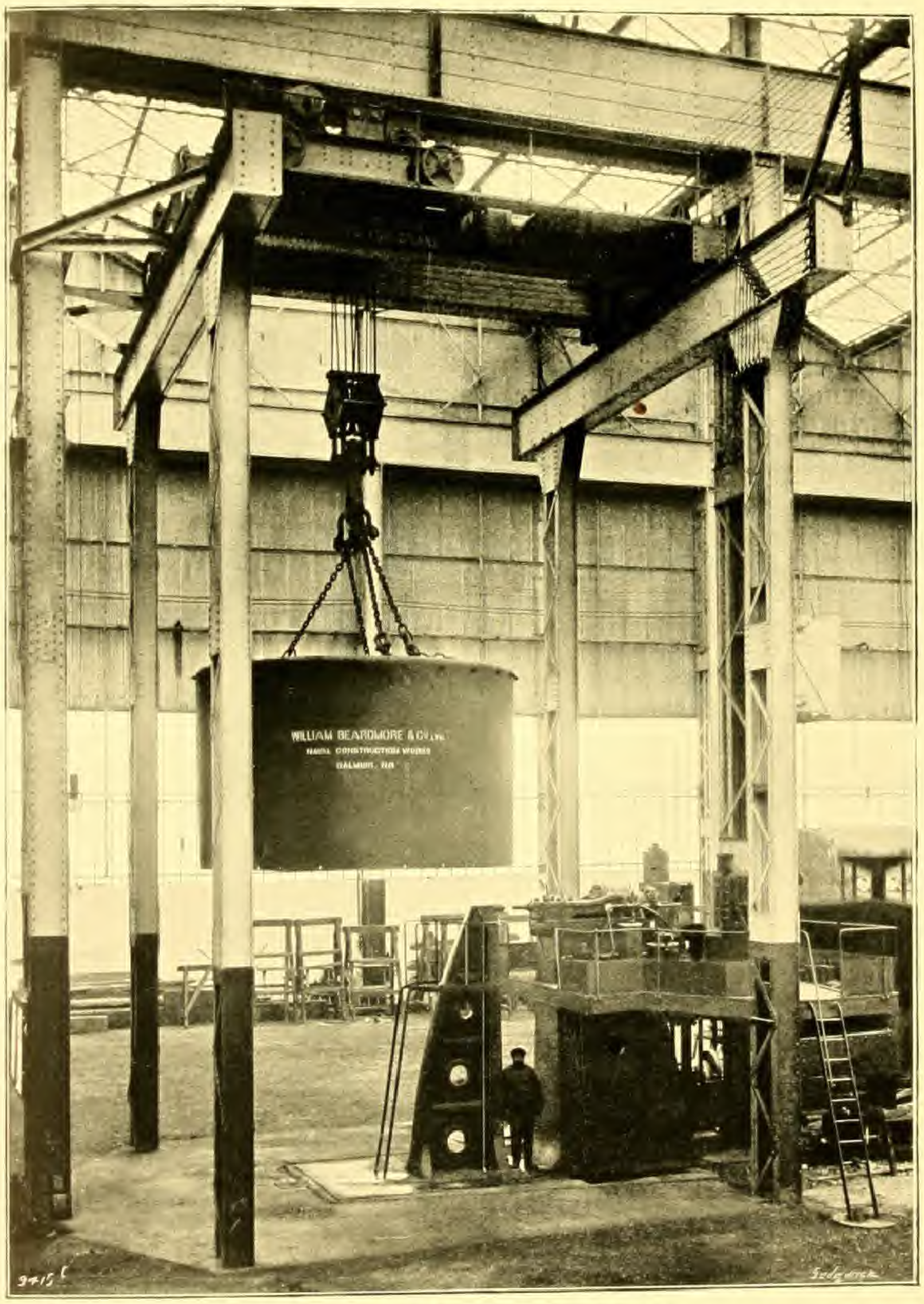
William Beardmore and Company's Machine Shop.

is nearly $5\frac{1}{2}$ acres. The bays extend from north to south, and the northern part of all five is utilised for boiler construction; while the southern part is arranged for the building of marine and other machinery. Here there has been erected the first large producer-gas engine completed for marine purposes.

All the materials for engines and boilers enter at the centre of the shops, those for engines being delivered to the south side of the line, and those for boilers to the north side. The machines in both departments are so disposed that the units during the process of manufacture travel from the delivery line towards the extreme end of the works. The completed engine leaves at the south end, and the boilers at the north end, on rails communicating with the fitting-out basin.

The bays range in width from 80 ft. to 50 ft., and the height in each case is 75 ft. The construction is very well shown in the engraving on the two preceding pages, and the general effect, with such an area of glazing, is particularly striking. In the shop there are thirteen cranes, all of the electric type, ranging from 60 tons downwards. Two of the 60-ton cranes can be yoked together to carry 120 tons, the columns and girders being made to suit the weight.

Suggestion is afforded of the immense height of the roof by the engraving on the opposite page, illustrating the riveting of the shell of a boiler in this department.



View in Beardmore's Boiler Works.

Vickers Sons and Maxim, Ltd.

THIS firm have not only an armour-plate and gun factory at Sheffield,[1] but a great arsenal at Barrow-in-Furness,[2] where there are constructed merchant steamers and warships, and all types of marine machinery, in addition to the extensive mountings for manipulating heavy guns and the projectiles for the same.[3]

Besides completing for war many of the most powerful British warships, including the battleships "Dominion" and "Vengeance," and a greater variety of other ships than almost any other firm, the Vickers have produced many fighting ships for foreign navies, and have helped to establish British credit as the greatest naval construction country in the world. Mention may be made of Togo's triumphant flagship, the "Mikasa"; the later Japanese battleship, the "Katori"; the battleship-cruiser "Rurik," for Russia; a battleship for Brazil, which promises to be equal to the best; and other fighting ships for Brazil, Chili, Peru, and other Powers.

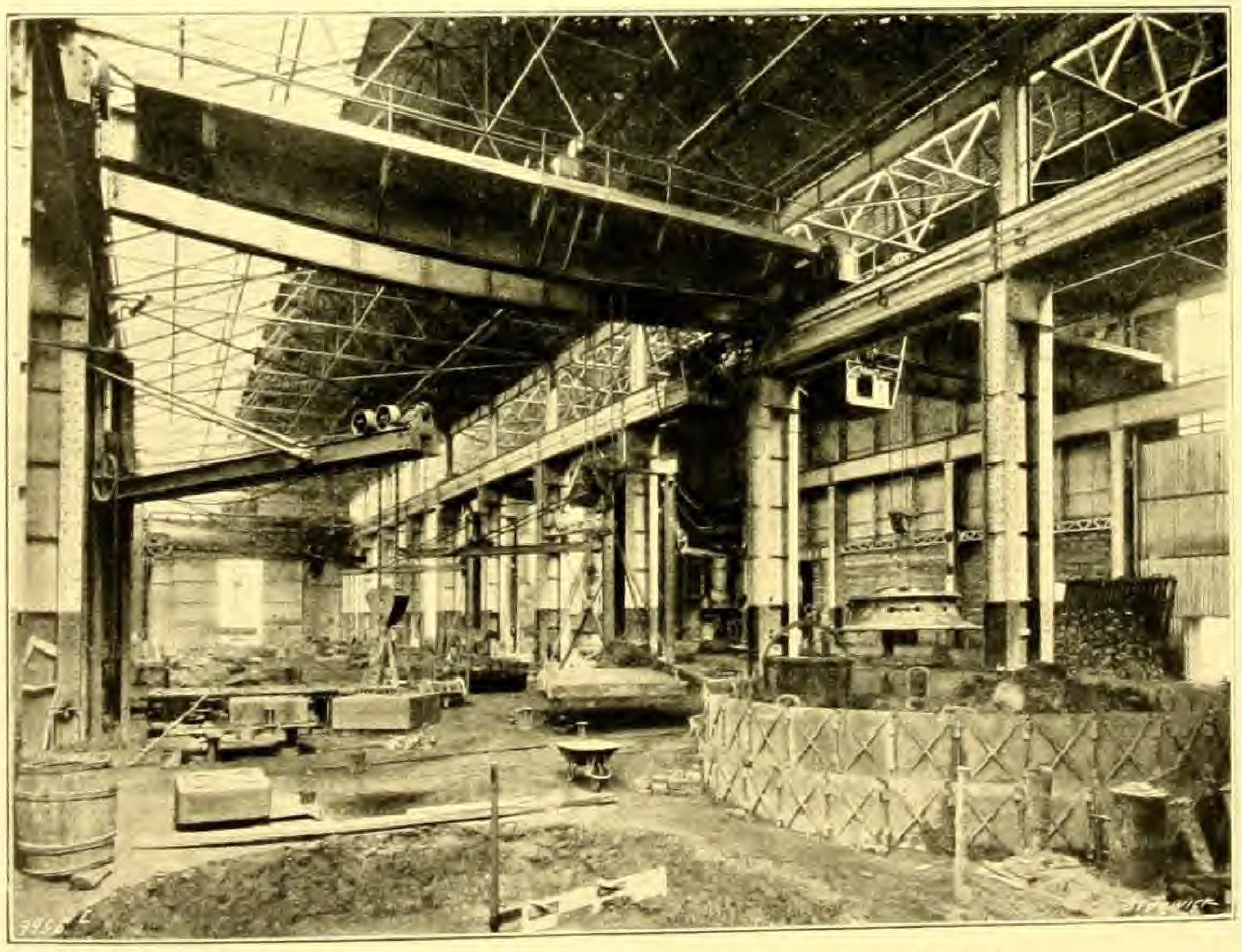
Sir William Arrol and Company, Limited, have built for the Vickers one or two notable workshops, including the iron foundry, which is illustrated on the opposite page. This is 264 ft. long and 150 ft. wide, the height of roof

[1] See ENGINEERING, vol. lxiv., pages 403, 430, 457, 521, 555, 583, 607, 639, 674, 703, 729, 760, 791.

[2] *Ibid.*, vol. lxxii., pages 169, 183, 215.

[3] *Ibid.*, vol. lxxii., page 110.

Vickers' Foundry for Marine Work at Barrow-in-Furness.



Vickers' Foundry for Ordnance at Barrow-in-Furness.

being 55 ft. There are fitted in it two 45-ton cranes and one of 25 tons, which suggest the massive character of the work turned out. In one bay marine castings are made, in the other ordnance parts are produced.

One of Vickers' Ordnance Workshops.

Another of the Arrol shops is that built in 1901, and illustrated on this page. This building is 250 ft. long, 175 ft. wide, and has a height of 30 ft. It is, it will be noted, devoted to light ordnance work.

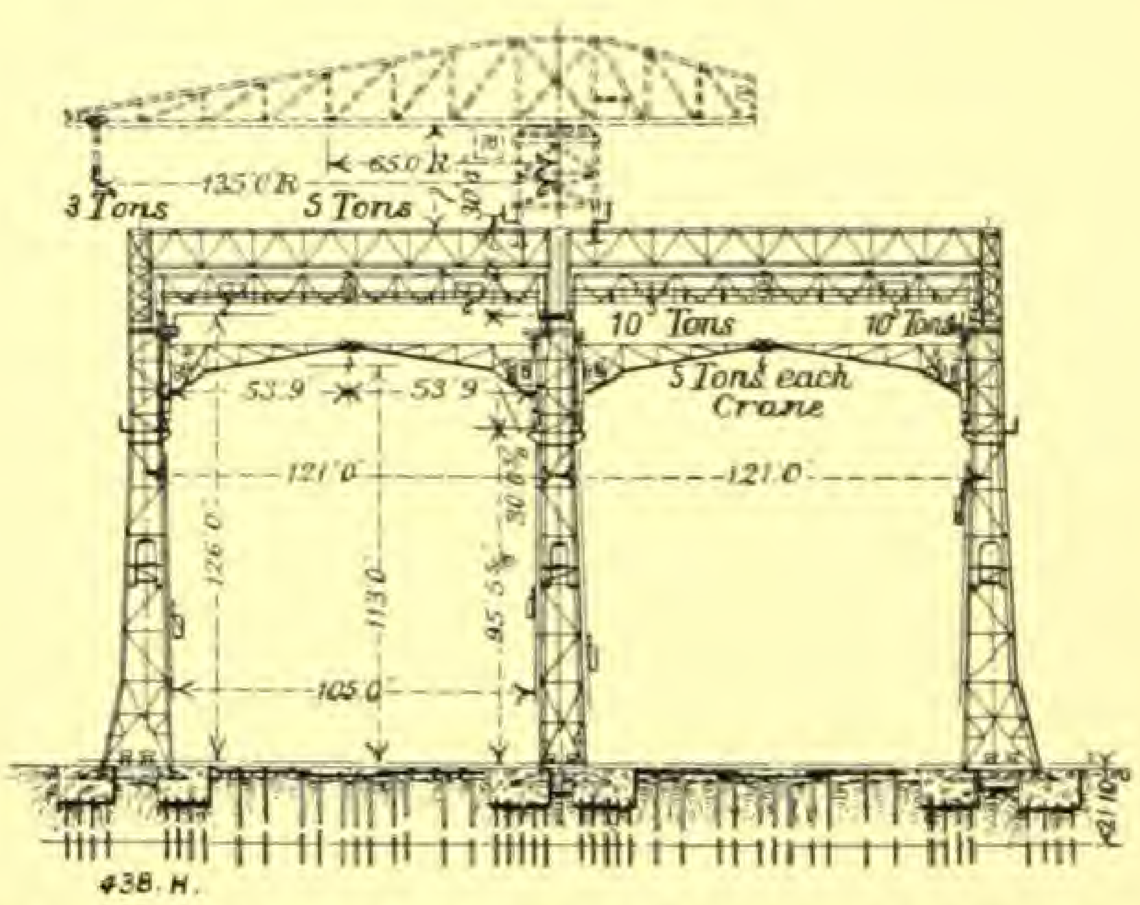
Shipbuilding Berth Equipment at Belfast.

THE firm of Messrs. Harland and Wolff have maintained a high reputation for enterprise in the equipment of their engine shops and shipbuilding yards at Belfast, and the developments of science and its practical adaptation to the construction of ships have been constantly kept in view by the firm to enable them to retain their prominent place among the builders of ocean liners. The keen rivalry on the Atlantic has resulted in the White Star Line placing an order with Messrs. Harland and Wolff for two immense steamers to be named the "Gigantic" and "Titanic," and as a consequence large additions have been made to the yard at Belfast to enable the firm to undertake the work. The principal addition has been a double gantry in the north yard, complete with cranes to deal with the erection and riveting of the hull and decks.[1] The new structure occupies the former site of three building slips, and is illustrated on the two following pages.

The structure, which has a total length of 840 ft., consists of three rows of towers placed 80 ft. apart longitudinally, and 121 ft. transversely. These towers support longitudinal girders tied together transversely at their tops, which in their turn carry a track over the centre row of towers, and on it a cantilever crane may travel the whole length of the structure.

[1] See ENGINEERING, vol. lxxxv., page 791.

A track is provided on the top of the longitudinal girders to carry travelling frames, which span across each berth. Each travelling frame contains two 10-ton travelling cranes. Another track is provided at the bottom of the longitudinal girders to carry 5-ton walking cranes. The rail level of the central track for the cantilever crane is 176 ft. above the ground level at the after end of the berths; the rail level for the travelling frames is 148 ft., and for the walking cranes 118 ft. above ground level.

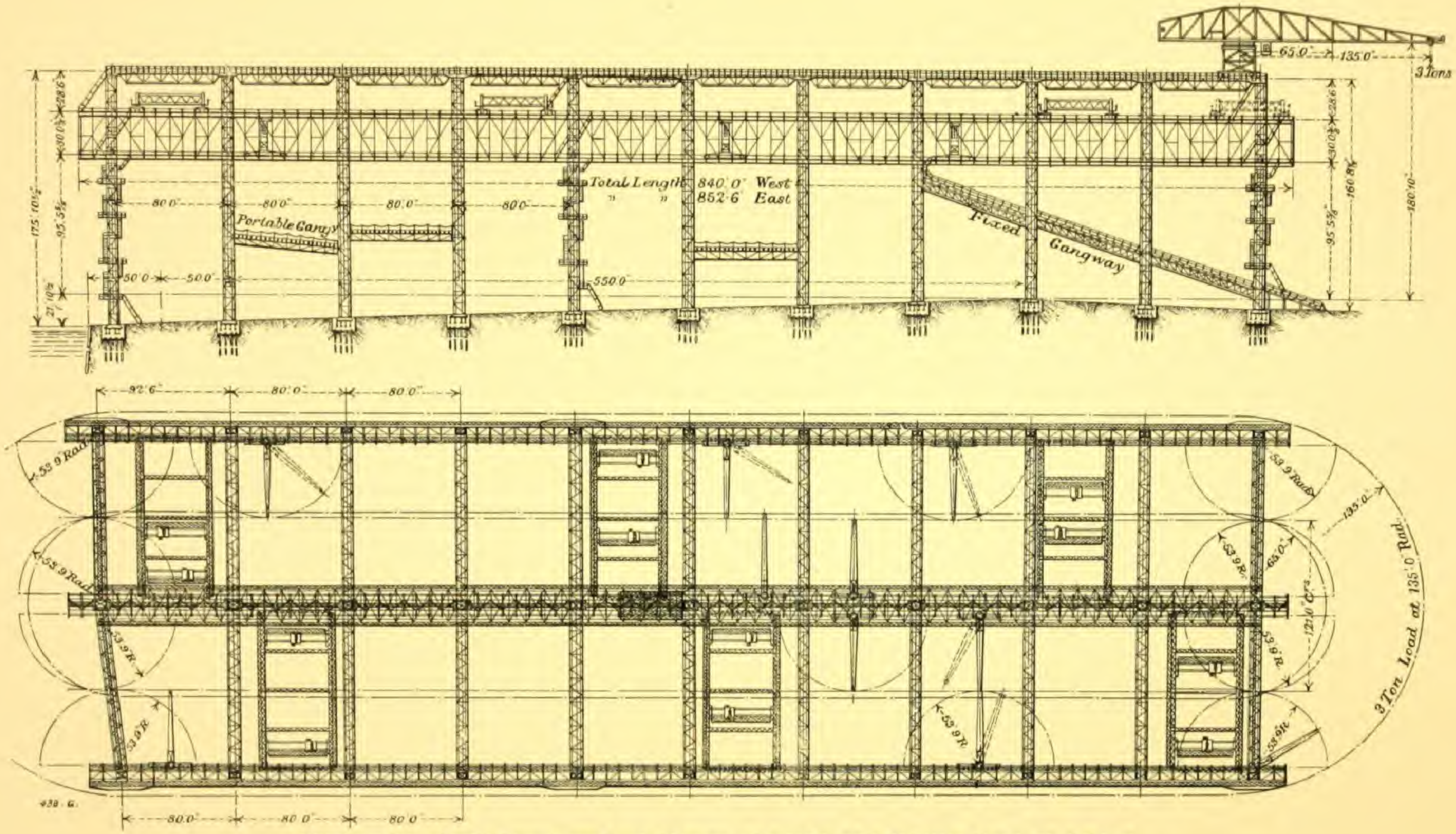


Section Showing Overhead Travelling Cranes.

The cantilever crane is capable of lifting 3 tons at a radius of 135 ft., and 5 tons at a radius of 65 ft., and has lifting, racking, slewing, and travelling motions. It commands a length of over 1050 ft. and a width of 270 ft., and can lift or deposit material at any point within this area.

The six travelling frames, provided with two 10-ton cranes within them, are for carrying the riveting machines. The internal travelling cranes have a longitudinal travel of about 35 ft. without the necessity of moving the frames. The frames at the after end of the berths have lifting eyes to deal with stern frames, etc., weighing up to 40 tons each.

The ten walking cranes are designed to lift 5 tons at a



Shipbuilding Berth Equipment at Messrs. Harland and Wolff's Yard at Belfast.

radius of 53 ft., and have lifting, slewing, and travelling motions fitted to them.

The inside faces of all towers are fitted with a special arrangement to carry portable riveting or building platforms at any level and at any angle to suit the line of plating. Fixed sloping gangways are provided at the forward end of the berths, and are arranged to pass through the towers. Easy access is provided by them to any level up to the underside of the longitudinal girders. A complete system of stairs and gangways is provided throughout the structure so that communication may be had to any part of the structure or to the cranes.

The cranes, which were supplied by Messrs. Stothert and Pitt, of Bath, as sub-contractors, are operated electrically, current being supplied from the main power station, and their collective power is 1600 horse-power.

The whole equipment was undertaken by Sir William Arrol and Company, Limited, and was designed by their engineering staff to the requirements of Messrs. Harland and Wolff.

Tranmere Bay Development Co., Ltd., Birkenhead.

THIS Company is now laying down what will be one of the largest shipbuilding and engineering establishments in the country.

The works are practically complete, and will have facilities for undertaking the largest class of vessels and machinery, including turbines.

The new buildings for the establishment have been designed, and are being built, by Sir William Arrol and Company, Limited.

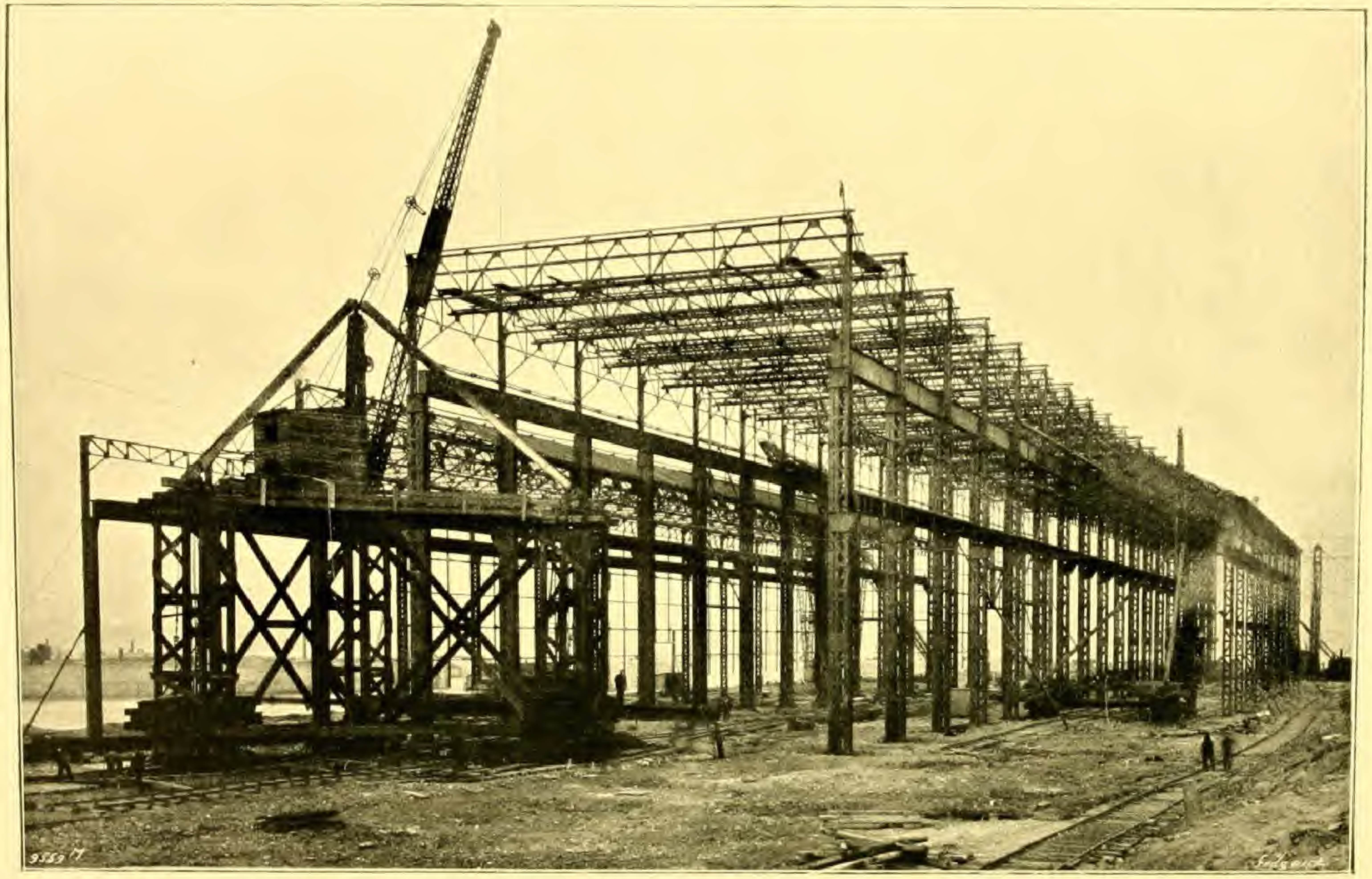
The engine erecting shop is 1075 ft. long and 77 ft. wide, with a height between the floor and the inside of the roof of 74 ft. There will be in this building 70-ton cranes, and the constructive steel work is so designed that two cranes close together may traverse the whole length of the shop with a load of 140 tons. This new shop is shown in course of erection on page 163.

At one side of the main shop there is a building 615 ft. long and 42 ft. 6 in. wide, with 35-ton cranes; and on the other a machine shop 525 ft. long and of the same width, also with 35-ton cranes.

The new boiler shop has a main bay 500 ft. long and 130 ft. wide, and 69 ft. high, with two 70-ton cranes, the structural steel work again being designed to lift and

traverse a load of 140 tons. On one side of this building there is a bay 420 ft. long and 50 ft. wide, and of a height of 38 ft.

In addition, Sir William Arrol and Company, Limited, are building a smith's shop 270 ft. long and 40 ft. wide, and several other buildings.



Building New Engine Works for the Tranmere Bay Development Company, Limited, Birkenhead.

The

Fairfield Shipbuilding and Engineering Company, Limited, Govan.

IT would be difficult to find in the extensive establishment of this well-known Company a workshop of primary importance with which Sir William Arrol and Company, Limited, have not been associated.[1] They designed and built the moulding loft of 275 ft. in length, where the lines of some of the best ships now afloat were laid down, including the immense battleship "Commonwealth," the 25-knot cruiser "Indomitable," and the new Canadian high-speed Mail steamers "Empress of Britain" and "Empress of Ireland." They constructed the joiners' shop, where some of the finest floating hotels have had their artistic furnishing completed. They erected engine and boiler shops, where powerful reciprocating and turbine machinery has been constructed; and brass foundries, fitting shops, mechanics' shops, smithies and brass finishers' departments, and others, have all been reconstructed by them within the past ten years.

As typical of the work done, we give engravings of the engine-erecting shop and the boiler shop. The former is illustrated on the opposite page. This new shop has a length of 291 ft. and a width of 55 ft., and the height to rail level is 50 ft., and to the roof ties 58 ft. The travelling

[1] See ENGINEERING, vol. l., pages 336, 393, 485, 599, 687.

The Engine Erecting Shop at Fairfield.

cranes range in capacity from 50 tons downwards. In this shop there have been constructed not only triple-expansion engines, ranging up to 15,000 horse-power, but many sets of Parsons turbine machinery for yachts, channel steamers, ocean liners, and high-speed cruisers. Amongst the turbine installations, prominence must be given to the 41,000 horse-power set for the four-screw cruiser "Indomitable," the 23,500 horse-power installation for the new battleship "Bellerophon," and turbines of 14,500 horse-power for two high-speed steamers for service between Marseilles and Egypt, to inaugurate a new British service in the Mediterranean.

The new boiler shop is illustrated on the opposite page. Here, not only are the largest of the cylindrical boilers constructed, but water-tube boilers for modern warships are completed. The length of the shop is 300 ft., the width 60 ft., and the height 55 ft. 6 in. There is a fine installation of cranes.

The Boiler Shop at Fairfield.

The

Coventry Ordnance Works, Glasgow.

THE new Coventry Ordnance Works at Scotstoun, Glasgow,[1] have been designed and built by Sir William Arrol and Company, Limited, for the manufacture of gun mountings for warships constructed by Messrs. Cammell, Laird and Co., Ltd., of Birkenhead; Messrs. John Brown and Co., Ltd., of Clydebank; and the Fairfield Shipbuilding and Engineering Company, Limited, Govan. These three shipbuilding companies are joint partners in this undertaking.

The works cover an area of 20 acres, and include two fine workshops, each having a length of 675 ft., and a collective width of 134 ft., the height being 63 ft.

In the larger bay, which we illustrate on the opposite page, there is to be an overhead traveller to carry a load of 100 tons, and others of from 60 tons capacity downward. The crane rail level is 44 ft. above the floor, so that the loads carried may not only be heavy but of large bulk. In the same bay, but at a lower level, there are rails for a series of 10-ton travellers for the ordinary shop work. An unusual feature in this bay is that both crane runways are the same span, so that the lighter cranes may be lifted to the higher level and worked there. The difficult problem of providing two runways in the same bay, and of the same span, at different levels was satisfactorily solved, as shown in the illustration.

[1] ENGINEERING, vol. lxxxiii., page 571

The Coventry Ordnance Works in Course of Construction.

Z

Parallel with these shops is a gantry of corresponding length—namely, 675 ft., with a span of 87 ft., and a height above the surface level of 50 ft. This gantry is for the accommodation of 100-ton and lighter cranes for the loading and discharging of gun mountings.

The design of roofing adopted in this case is the "umbrella" or "ridge-and-furrow" type, supported in the ridge, and entirely covered with glass. The roof is carried on cross-girders at 29 ft. centres, which rest upon columns with deep foundations. The advantage of this system of roof construction is that the building will be more stable than one of the ordinary ridged roof type, under the severe racking stresses due to the heavy high-speed travelling cranes.

The whole of the constructional steel work of these large shops, requiring the use of 2500 tons of steel, was completed within six months of the date of the placing of the order. Of this time sixteen weeks sufficed for the erection of the structure — a proof of the splendid organisation and suitability of the manufacturing and erecting plant employed by the Company.

Glenfield and Kennedy, Ltd., Kilmarnock.

THIS firm, which has a high repute for hydraulic machinery, water meters, and foundry work gene-

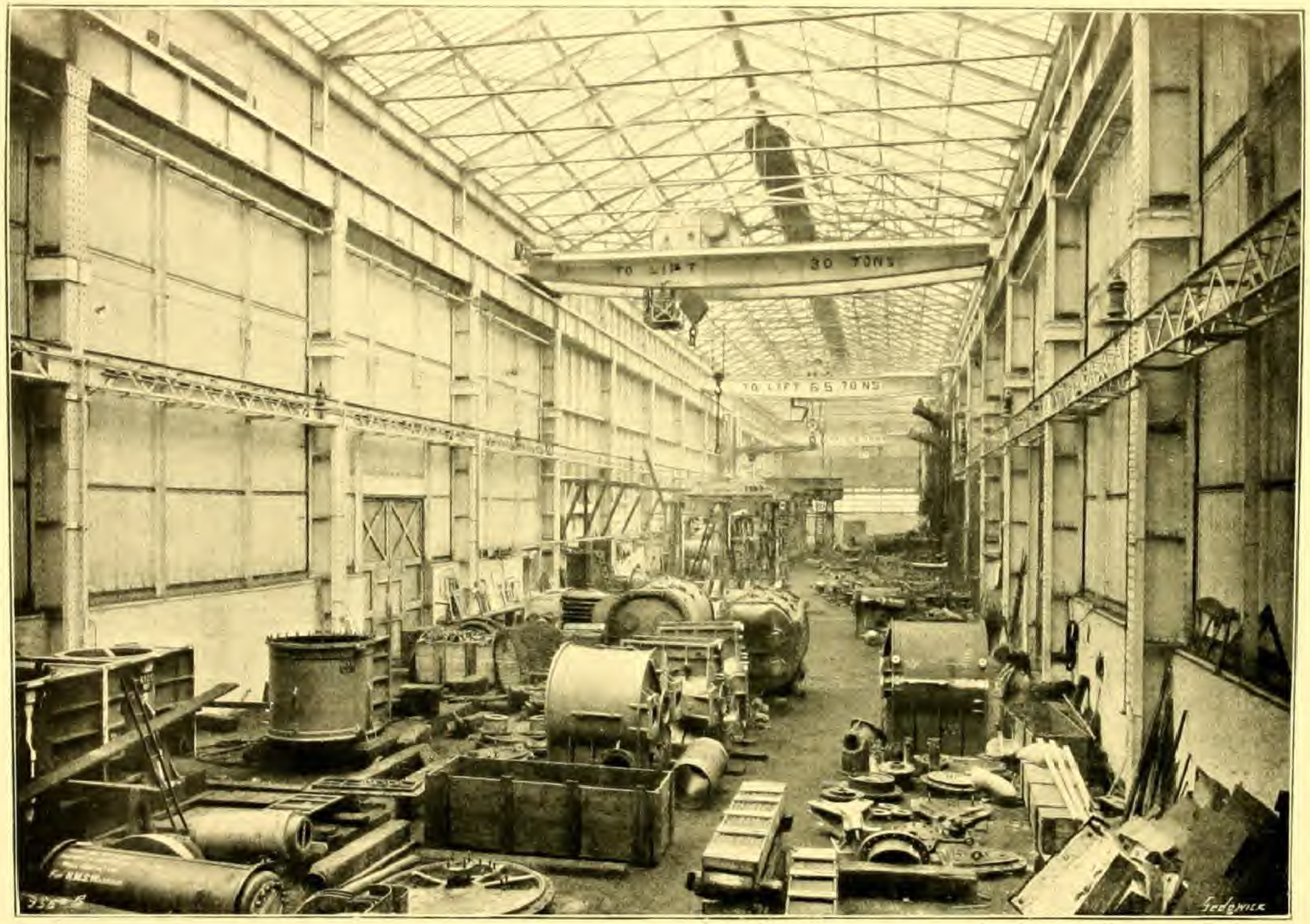
rally, has had two foundries constructed by Sir William Arrol and Company, Limited, one of which is illustrated on this page. The length is 251 ft. and the width 137 ft., with a height of 44 ft. There has also been erected a foundry for light work. This is of the same length, of greater width, and of a height of 67 ft.

The Wallsend Slipway and Engineering Company, Limited, Wallsend-on-Tyne.

FEW engineering establishments in this country, or even abroad, have experienced so rapid an advance, alike in volume and quality of work done, as that of the Wallsend Slipway and Engineering Company since Mr. Andrew Laing became associated with the establishment.

Previously the largest set of machinery completed was of 4,600 horse-power, and the average output of machinery per annum was 26,000 indicated horse - power, with a maximum of 40,000 indicated horse-power. During the past five years the average has been 67,000 horse-power with a maximum of 115,000 indicated horse-power. The largest set of machinery—that for the 25-knot Cunard liner "Mauretania"—is of 70,000 horse-power. At the same time the firm have been entrusted with machinery for several warships, including the turbines for the new battleship "Superb," of 23,000 shaft horse-power. Thus the firm is in the front rank in connection with turbine machinery for warships and merchantmen.

The new buildings have been designed and constructed by Sir William Arrol and Company, Limited, and were, as a rule, built outside and around the old shops, so as to involve the minimum of inconvenience during the process of erection. We illustrate two of the



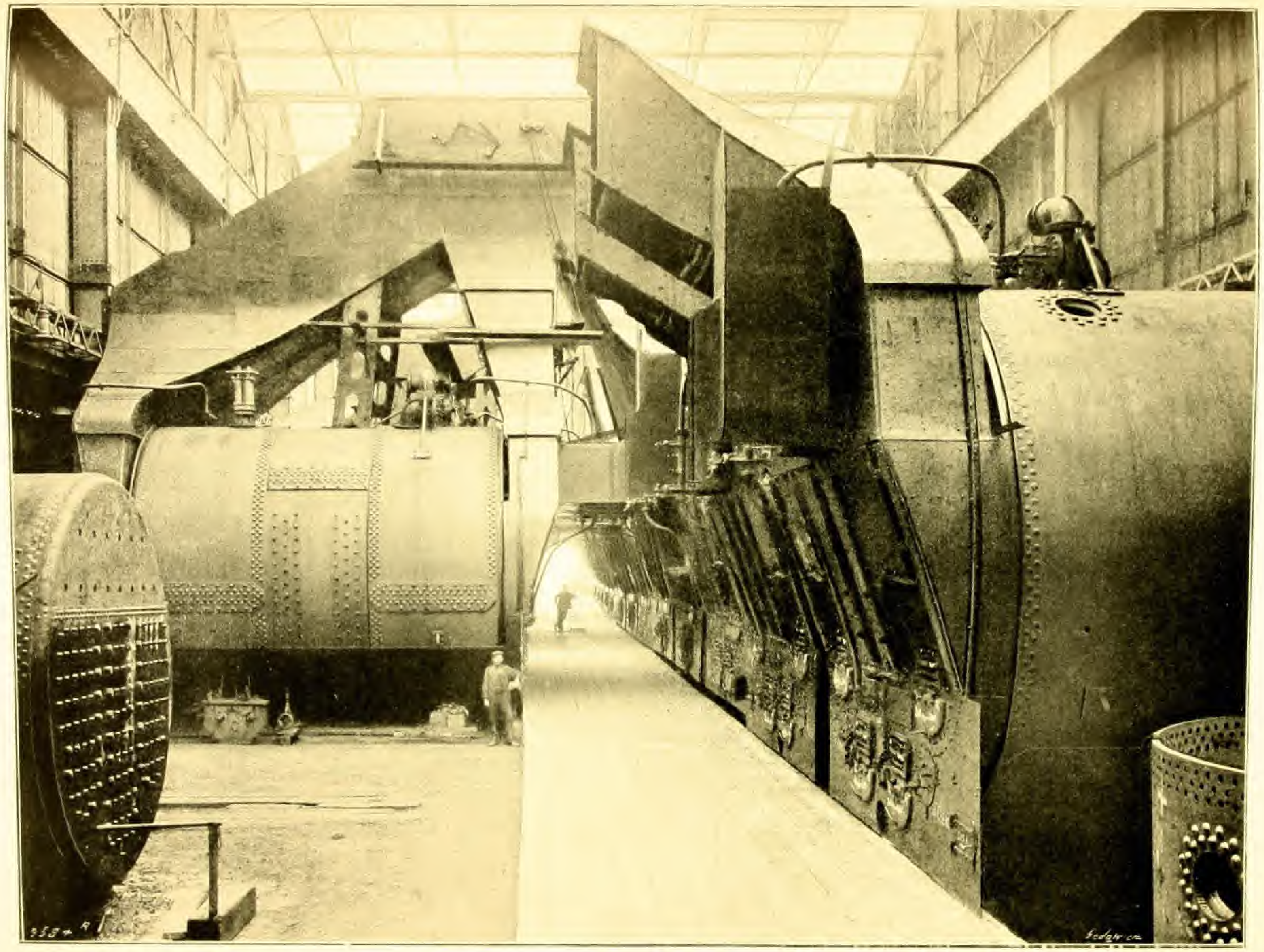
Engine Works of Wallsend Slipway and Engineering Company, Limited.

new buildings—the engine-erecting shop on the preceding page, and the boiler shop on the opposite page.

The erecting shop, in which the immense turbines for the "Mauretania" were completed, has a length of 570 ft., but is built so that when required an extension can easily be made. The width is 60 ft., and the height 65 ft. In this shop there are, as shown in the engraving, two cranes of 65 tons and one of 30 tons capacity.

The boiler shop, which is illustrated on the opposite page, has a length of 330 ft., a width of 75 ft., and a height of 70 ft. A proof of the capacity of this shop is afforded by the fact that the output of marine boilers is almost one per week. As to its sufficiency alike in breadth and height, there is the fact that the whole of the 25 boilers for the "Mauretania" were erected in this shop, and their uptakes, steam pipes, and platforms completed, as shown in our engraving.[1] In this way all the parts were fitted together, so that when marked and removed to the ship the final fitting and riveting was most expeditiously done.

[1] See ENGINEERING, vol. lxxxii., page 349.



Boiler Shop of Wallsend Slipway and Engineering Company, Limited.

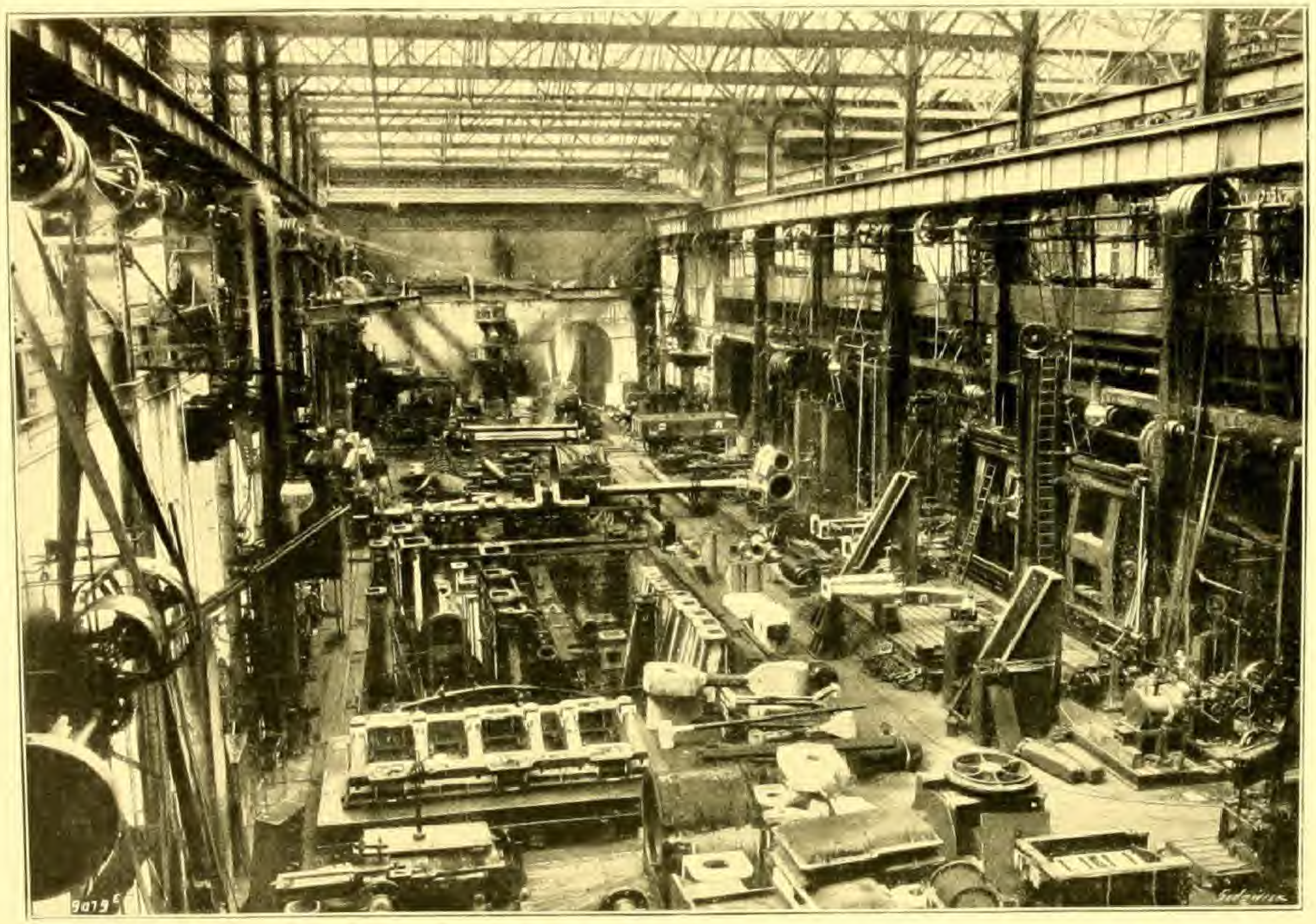
Scotts' Shipbuilding and Engineering Company, Ltd., Greenock.[1]

ONE of the earliest shops constructed by the Company was for the Scotts' Shipbuilding and Engineering Company, Ltd., of Greenock, and the illustration on the opposite page is, from this point of view, of special interest.

In this shop some of the finest modern marine machinery has been completed, including engines of 27,000 indicated horse-power, for the 24-knot armoured cruiser "Defence." This is the latest of the long succession of naval contracts carried out by the Scotts' Company, whose association with the Admiralty began as far back as 1803, when they built the sloop of war "The Prince of Wales," succeeded by the first Clyde-built steam frigate and by many notable warships.

Indeed, Scotts' Company are almost unique, as for two centuries they have taken a prominent part in the development of shipbuilding and marine engineering; and the works have been carried on by a succession of Scotts in the direct line of descent, the present managing directors belonging to the sixth generation.

[1] See ENGINEERING, vol. lxxxi., page 171.



Engine Fitting Shop of Scotts' Shipbuilding and Engineering Company, Limited, Greenock.

Yarrow and Co., Ltd.

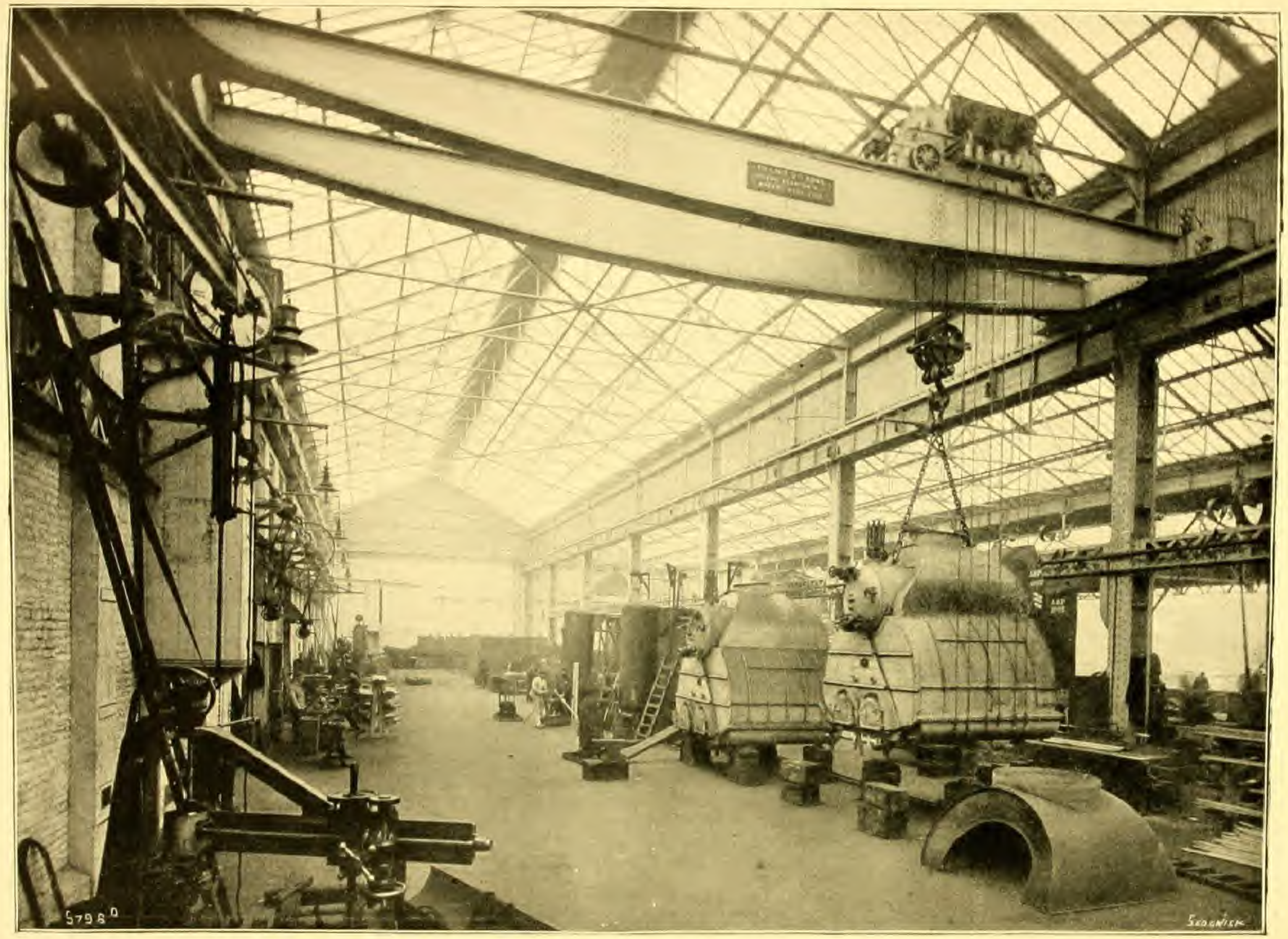
WHEN the story of the development of the torpedo boat and of light high-speed machinery is written, the name of Yarrow will occupy a prominent place. The firm have devoted a large amount of time and money to the solving of problems associated with the design of such craft.

In the first place, they tackled the question of the tensile strength of the material used, with the result that they were among the first to advocate a high tensile steel. The method of riveting this material was also the subject of careful experiments.

Research work also resulted in the development of the Yarrow water-tube boiler, now so extensively adopted for all types of warships. The horse-power of Yarrow boilers constructed by the firm up to this date exceeds 800,000, and the output of their licencees in this country and abroad is about double this total.

In connection with high-speed propelling engines for torpedo craft, with petrol engines for motor boats, etc., and later with turbine machinery, the firm have done original work, particularly in connection with the reduction of vibration, the efficiency of screw-propellers, and the influence of depth of water upon the speed of ships.

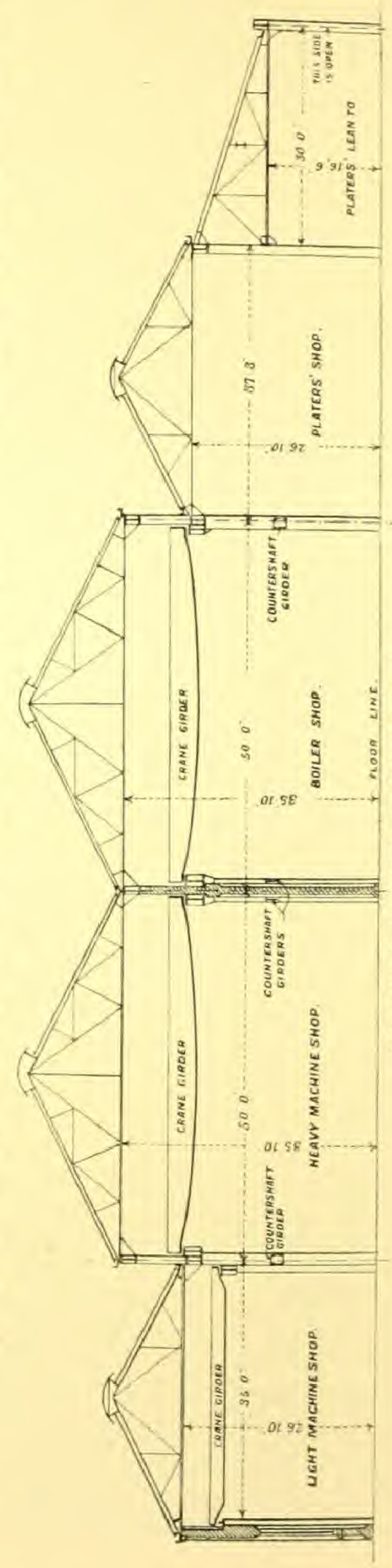
It will be readily understood that a firm which attacks from a scientific standpoint all such mechanical problems will attach great importance not only to the suitability of its manufacturing plant, but also to the design of its workshops.



Boiler Shop at Yarrow's Works, Poplar.

Some years ago they entirely reconstructed their works at Poplar,[1] and Sir William Arrol and Company, Limited, designed and built for them at that time a boiler shop, platers', and machine shop, having a length of 390 ft., and of a breadth and height shown on the annexed section. These works are illustrated on pages 179 and 181.

In the course of time, however, owing to the great expense of construction, due to the excessive rates and to the high cost of labour in the East end of London, Messrs. Yarrow were forced to remove to a district offering more economical conditions, and a site on the north bank of the Clyde, three or four miles west of Glasgow, was chosen. There, at Scotstoun, entirely new works have been built,[2] and as a consequence of experience of Sir William Arrol and Company's work, the construction of all the shops was entrusted to them without even a formal contract.



CROSS SECTION THROUGH MAIN SHOPS (Looking North)

Yarrow and Company's Works at Poplar, London.

[1] See ENGINEERING, vol. lxxi., page 441.

[2] See ENGINEERING, vol. lxxxi., page 353; vol. lxxxiii., page 571.

Heavy Machine Shop at Yarrow's Works, Poplar.

The total area of the new works is $12\frac{1}{2}$ acres, but the Company have purchased for extension an equal area of vacant land immediately to the east of the new site. On the area occupied, which has a frontage to the river of 750 ft., with a depth of 700 ft., there is being constructed a fitting-out basin 320 ft. long and 85 ft. wide, set at a slight angle to the flow of the river, so as to facilitate the entrance and exit of vessels.

To the east of this there are eight building berths, at the head of which the platers' shed has been constructed. The carpenters' and pattern-makers' shop and the smithy are to the west of the basin.

On the landward boundary, on the west side of the entrance, there is the machine shop, illustrated on the opposite page. The total length is 248 ft., and there are three bays, of a total width of 155 ft. 6 in. To the east of the entrance there is a boiler shop of 303 ft. in length, with three bays of a total width of 153 ft. The offices are located between these two shops, with the entrance, close by which the railway siding passes.

One of the new features is the construction of a yard gantry, 330 ft. long, with a height above the ground level of 26 ft., for the accommodation of a 7-ton electric crane of 85-ft. span. The space below this gantry is to be used for building shallow-draught steamers for shipment in pieces.

The fitting-out or tidal basin is completely covered over by a roofing, entirely glazed, carried on columns 92 ft. apart transversely, and 55 ft. longitudinally. The whole area of this basin is commanded by a 50-ton electric crane.

The total width of the building over the tidal basin is 140 ft., consisting of the main gantry building and two

Yarrow's New Engine Works at Glasgow in Course of Erection.

lean-to roofs at the sides. The workmen engaged on the wharf, or aboard the vessels, will thus be protected from weather.

The whole of the work will thus be carried on under roof, except that done on the building berths.

Yarrow's New Boiler Shop at Glasgow in Course of Erection.

Parsons Marine Steam Turbine Company, Ltd., Wallsend-on-Tyne.

THE buildings within which so many of Parsons marine steam turbines have had their origin were constructed by Sir William Arrol and Company, Limited. This new prime mover has become, in a comparatively few years, one of the most largely adopted engines for high-speed ships.

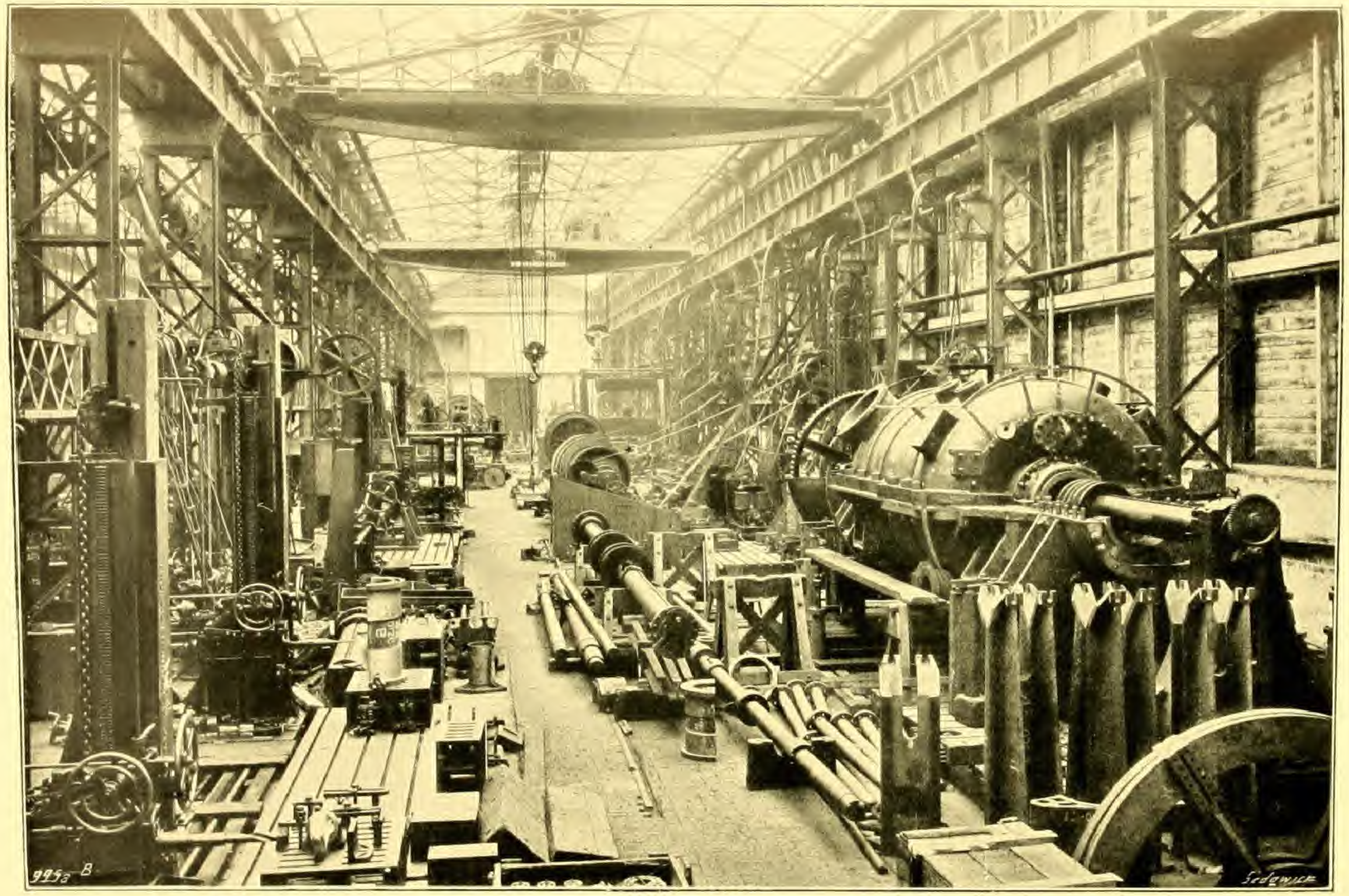
As indicating the progress of the turbine, it may be stated that whereas in the beginning of 1900 there was only one vessel driven by this engine, the power of marine turbines in use increased to 70,000 shaft horse-power at the end of 1903, to 150,000 horse-power in 1904, to 270,000 horse-power in 1905, and to 390,000 horse-power at the end of 1906. The total horse-power of Parsons marine turbines completed and on order is now 1,250,000, including the machinery being built by the licensees and by the Parsons Company.[1]

The Company constructed new works at Wallsend-on-Tyne in 1899, where the principal machine shops were erected by Sir William Arrol and Company, Limited, including, as noted in Table II., page 144, erecting shops, pattern shops, foundries, smithies, test-houses, etc.[2]

On the opposite page there is reproduced an engraving of the erecting shop, which is 385 ft. long and 80 ft. broad, with a height of 44 ft.

[1] See "Marine Steam Turbine Development," by Hon. C. A. Parsons and R. J. Walker, in the Proceedings of the North-East Coast Institution of Engineers and Shipbuilders, March, 1907.

[2] See ENGINEERING, vol. lxviii., pages 191, 221, 255.



Parsons' Marine Turbine Machine Shop.

Babcock and Wilcox, Ltd., Renfrew.

OUR next illustration shows the machine shop of Messrs. Babcock and Wilcox, Ltd., the constructors of a type of water-tube boiler now very largely adopted in the Navy and merchant service. The advance in favour of this boiler is indicated by the following Table, showing the horse-power of the boilers manufactured at the Company's Works at Renfrew and elsewhere, and by other firms under license of the Company :—

——	Number of Ships.	Number of Boilers.	Indicated Horse-power.
Up to end of 1889	1	1	275
1890 to 1892	3	3	950
1893 to 1895	6	10	6,150
1896 to 1898	41	84	56,345
1899 to 1901	75	302	283,925
1902 to 1904	78	528	564,631
Total to March, 1907	266	1,268	1,293,715

At the Renfrew Works various shops have been constructed, the latest being the marine shop illustrated on the opposite page. It is 350 ft. long, with a width of 216 ft.

Marine Boiler Shop at Babcock and Wilcox, Limited, Renfrew.

David Rowan and Sons, Glasgow.

MESSRS. D. ROWAN AND SONS have occupied a favourable position amongst the Clyde engineers almost since the beginning of the steam era. In recent years, under the superintendence of the late Mr. James Rowan, the son of the founder, and of his partner, Mr. William Thomson, the works were entirely reorganised,[1] to make the most of the available area, which is especially valuable, as the establishment is located in a fairly crowded part of Glasgow. At the same time an admirable system of management was instituted. It was in connection with this rearrangement that the firm modified the premium system of wages, which has been widely applied. This system was described in papers read at the Engineering Congress held in Glasgow in 1901.[2] But here we are concerned only with the machine shops.

The boiler department is 234 ft. long, the erecting shop being of 90 ft. span, and the light plating shop of 60 ft. span. The roof, it will be seen, is light in structure, and affords abundant natural lighting.

The 90-ft. span is served by three travellers, which extend the full width, and have a capacity two of 40 tons and the other of 25 tons. The work done is for merchant steamers, and experience has shown that the great majority of the loads for the heavy crane consist

[1] See ENGINEERING, vol. lxxiii., page 597.

[2] See ENGINEERING, vol. lxxii., pages 351, 379, 383.

Boiler-Erecting Department, Looking North. D. Rowan and Sons, Glasgow.

of the shells of single-ended marine boilers weighing from 18 to 20 tons. In the light plating department the overhead cranes are of 10 to 20 tons capacity.

The view on the preceding page illustrates the boiler-erecting department looking north; while on the opposite page there is a view of the east bay, where light plating work is done.

In addition, two engine shops were constructed, the larger and later of 220 ft. in length and 57 ft. span, with a height of 57 ft. Details of the various buildings by the Company for this firm are given on the Table on page 143.

Boiler Shop, East Bay. Light Plating Department. D. Rowan and Sons, Glasgow.

G. and J. Weir, Ltd., Cathcart.

WEIR'S pumps, Weir's feed-heaters, and one or two other auxiliaries for marine and land machinery, enjoy a universal reputation, not only because of the excellence of design, but because absolute reliability is ensured as a result of specialised manufacture. This is a consequence of progressive management, and the works of the firm at Cathcart[1] are not only well planned and well equipped but especially well lighted.

The principal machine shops have been constructed by Sir William Arrol and Company, Limited, and on the opposite page are two illustrations, the one showing the fitting shop and the other the foundry. The first-named is a building 372 ft. long and 41 ft. wide, with a height of 43 ft., and along it there travels an overhead crane of 30 tons capacity.

The foundry is 210 ft. long, with a width of 105 ft., and a height of 44 ft. The photograph reproduced is remarkable as illustrating the effect of the extensive glazing of the roof. There can be no doubt that with good lighting the work is more expeditiously and more accurately carried out.

[1] See ENGINEERING, vol. lxxi., page 795.

Fitting Shop. G. and J. Weir, Limited.

Foundry. G. and J. Weir, Limited.

Stewarts and Lloyds, Ltd.

THIS firm is probably the best-known iron and steel tube manufacturing concern in the world, representing, as it does, the amalgamation of the two largest producers in this country. The illustration on the opposite page shows the several bays of an extensive shop which was designed and built by Sir William Arrol and Company, Limited, in 1906.

This building, which is 600 ft. long and of a total width of 480 ft., has a height of 36 ft. It is part of the Imperial Works at Coatbridge, N.B., and the view shows that notwithstanding the smoke which is usually associated with the work of tube making, a light and clear atmosphere is possible when some effort is made to achieve it.

Imperial Tube Works, Coatbridge, of Stewarts and Lloyds, Limited.

Neilson, Reid, and Co., Ltd., Glasgow.

(Now North British Locomotive Company, Ltd.)

THIS firm, which has recently been formed with others into the North British Locomotive Company, Ltd., is one with a splendid historical record, having constructed locomotives, not only for many of the British lines, but for nearly every country in the world. It has thus assisted materially towards the maintenance of British prestige in this department of mechanical engineering.

During the last decade of the nineteenth century very extensive reconstruction works were carried out at the firm's works at Springburn, Glasgow, and several new shops were built by Sir William Arrol and Company, Limited.

A representative erecting shop is illustrated on the opposite page. The length is 706 ft., the width 44 ft., and the height 54 ft. It is traversed by three cranes, one of which has a capacity of 75 tons. The view offers a suggestion of the extensive character of the work carried out by the firm, owing to the presence of so many locomotives, that in the foreground being one of several compound engines constructed for the Dutch railways.

Still later, new works have been constructed at Polmadie for the North British Locomotive Company, Ltd. These cover an area of about two acres, and the dimensions of the buildings are given on the Table on page 144.

Erecting Shop of Neilson, Reid, and Company (now North British Locomotive Company, Limited), Glasgow.

John Spencer and Sons, Ltd., Newburn.

OUR next engraving is a view of one of the latest machine shops built for the Newburn Steel Works of Messrs. John Spencer and Sons, Ltd., an establishment founded in 1810 by John Spencer. He was a maker of files in works at Newcastle, and these were carried to market on the backs of donkeys.

Later, Mr. Spencer concluded that it would be profitable to make his own steel, and a converting furnace was accordingly laid down at Newburn, to which site he had been attracted by the available water-power, which was used for driving the train of rolls for forming the steel. In 1845 a steam engine was purchased, and was the first engine used for driving a train of rolls directly. Spring-making and other units for railway stock were early products of the firm, which has developed until it now occupies a prominent place among the producers of steel forgings for heavy marine and other machinery.[1]

The fitting shop, erected by Sir William Arrol and Company, Limited, illustrated on the opposite page, has a length of 335 ft. and a width of 77 ft., with a height of 51 ft. The electric cranes fitted range up to 50 tons lifting capacity.

[1] See ENGINEERING, vol. lxxiv., page 134.

Newburn Steelworks of J. Spencer and Company, Limited.

Marshall, Sons, and Co., Ltd., Gainsborough.

As representing another class of engineering factory, we reproduce on the facing page an engraving of a new boiler shop built by the Company for Marshall, Sons, and Co., Ltd., of Gainsborough. This establishment has long been identified with general engineering and millwright work, while the manufacture of boilers of all types has for many years been one of the prominent successes of the firm.

The works were originally founded in 1848, when a small engineering and general millwright's business in Gainsborough was purchased by Mr. William Marshall. Operations were at first conducted on a very small scale, but eventually $1\frac{1}{2}$ acres of land were purchased, and on this the nucleus of the present works was founded in 1855-56.

The business has since grown to enormous proportions, and over 120,000 engines, boilers, etc., have been turned out of the Britannia Works.

The new boiler shop, which is illustrated on the opposite page, has a length of 400 ft. and a width in three bays of 175 ft., the height being 56 ft. There are eleven overhead cranes, ranging from 30 tons to 5 tons.

Works of Marshall, Sons and Company, Limited, Gainsborough.

The Charing Cross, City, and West End Electric Company, Bow Station.

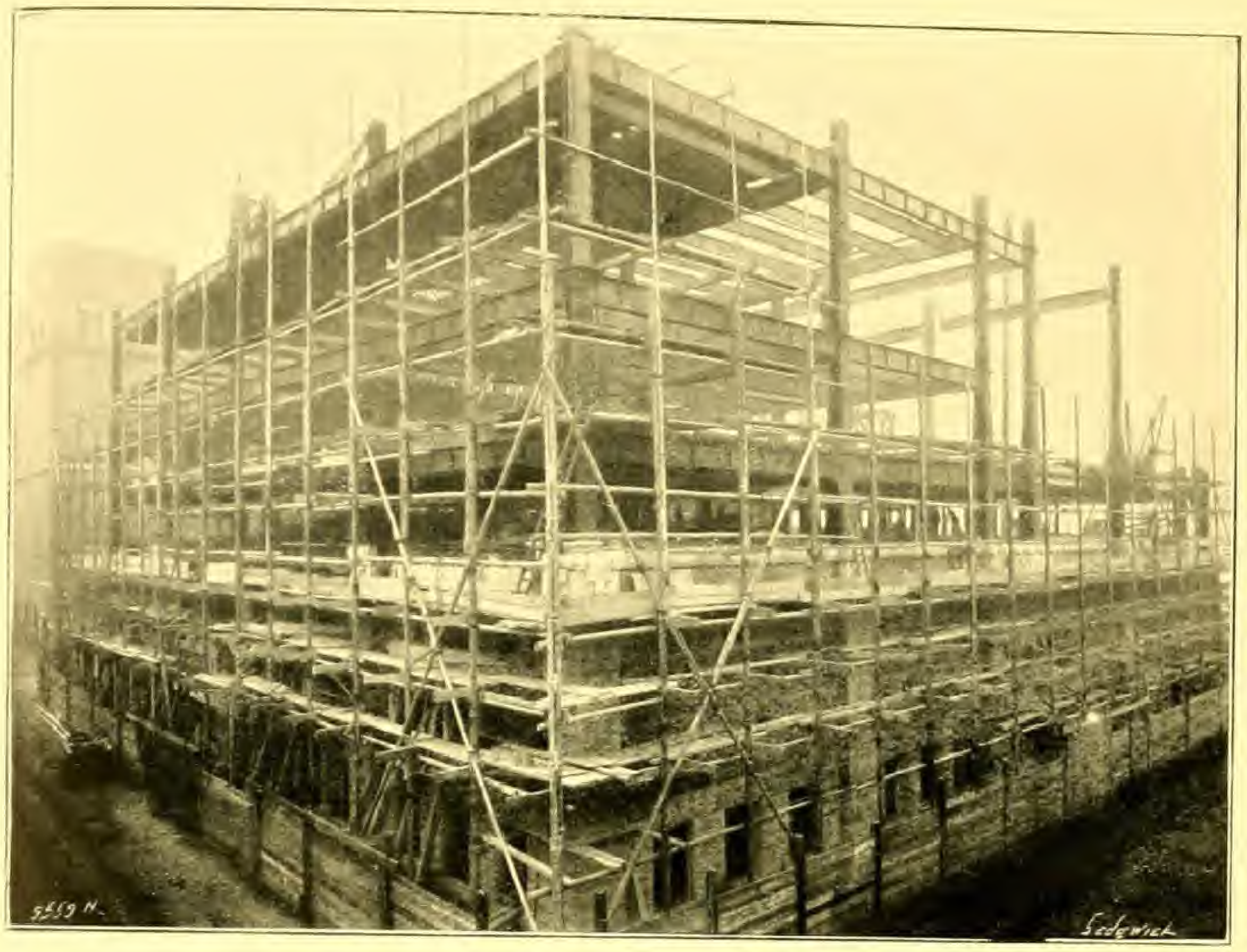
BY way of variety, our next illustration is a view of an electric-power station for this well-known Metropolitan Company. Sir William Arrol and Company, Limited, designed and built a boiler house 300 ft. long, with a width of 77 ft. and a height of 93 ft., and an engine house, 300 ft. long, with a width of 76 ft. and a height of 93 ft., the latter having an overhead crane of 30 tons capacity. This was for the Bow Station of the Company.[1]

[1] See ENGINEERING, vol. lxxxi., pages 63 and 96.

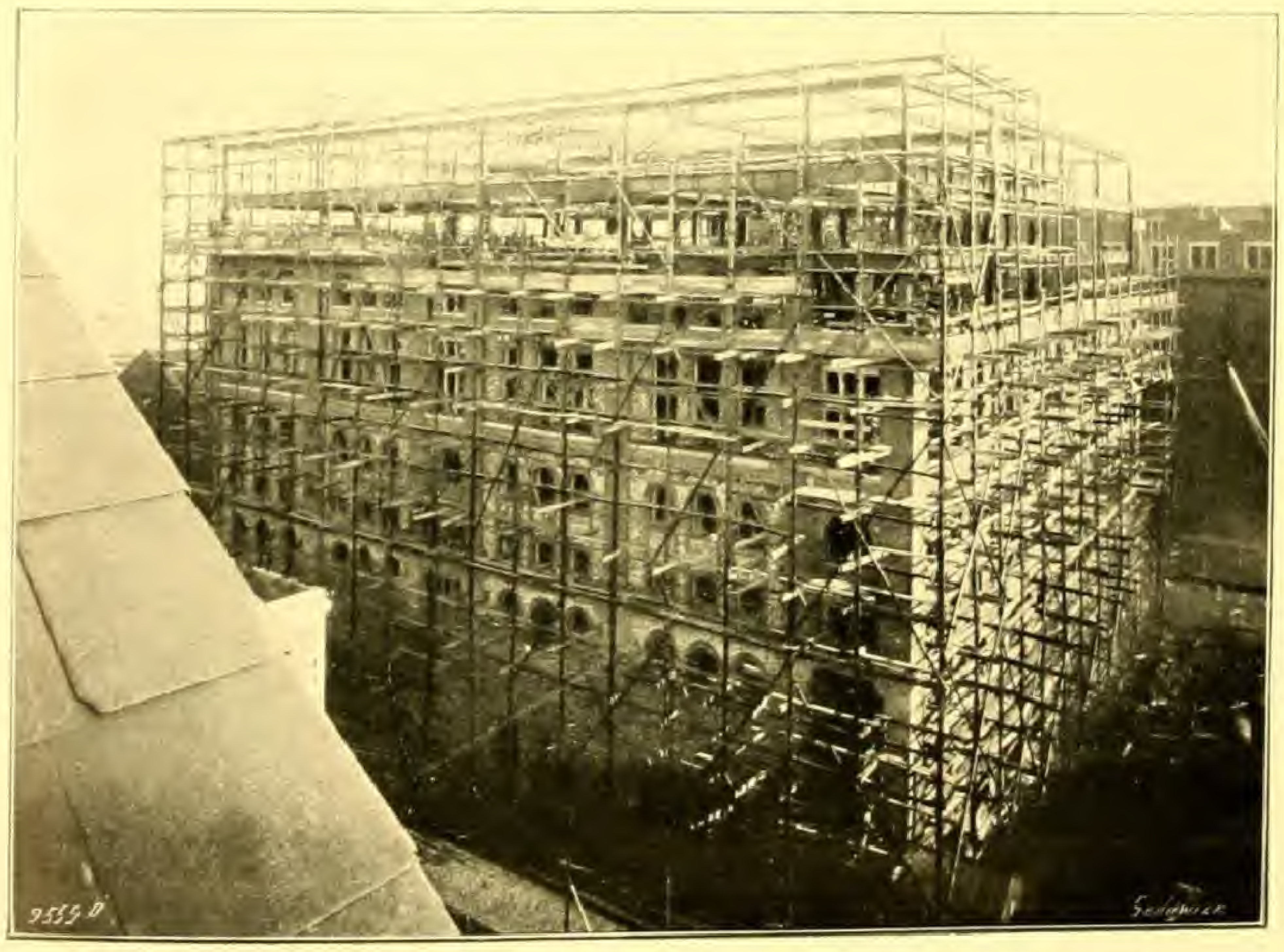
The Bow Electric Generating Station.

A. Guinness, Son and Co., Ltd., Dublin.

THE two engravings on the opposite page illustrate stages in the process of erecting a steel building for this famous firm of Dublin brewers. The building is 165 ft. by 146 ft., and the total height is 116 ft. As will be seen, it is constructed of built-up square stanchions, with plate girders and joists, the outer covering being of masonry. The work was done very expeditiously, the time occupied being twenty months from the date of signing the contract.



Storage Building at Guinness's Brewery in Course of Construction.
1st January, 1904.



Storage Building at Guinness's Brewery in Course of Constructiou.
3rd May, 1904.

Chimney Stack.

ANOTHER variety of constructional steel work is illustrated on this page, in the form of a steel chimney

stack, constructed at Guinness's Brewery in Dublin. It has a total height of 172 ft. from ground level.

MECHANICAL ENGINEERING.

Experience and its Application.

THE success of the large bridge and other constructional works undertaken by Sir William Arrol and Company, Limited, is due not alone to design, but in part to ingenuity in devising suitable engineering appliances and to foresight in providing against difficulties in manufacture and erection. The varied necessities of each case involve almost continuous research and invention, and a consequence is the production of efficient machinery of all kinds.

The beginning of the now extensive engineering section of Sir William Arrol and Company's Works dates practically from the evolution of the hydraulic riveter, when Sir William Arrol was compelled to enter upon the experiments which brought success by the workmen going on strike during the building of one of the early bridges of large dimensions. This use of hydraulic power led to a closer study of pumps, and many installations have been manufactured. The making of presses, cranes, and special machine tools followed, and air compressors, air locks, and other appliances for bridge pier and shaft sinking were soon added.

Experience in the use of such appliances is, in all cases, carefully collated; and thus from time to time improvements have been effected, so that the mechanical engineering productions of the Company have attained a high degree of efficiency.

In the succeeding pages there are illustrated some of these productions; many of them are in use in the principal works in the country.

Hydraulic Pumps.

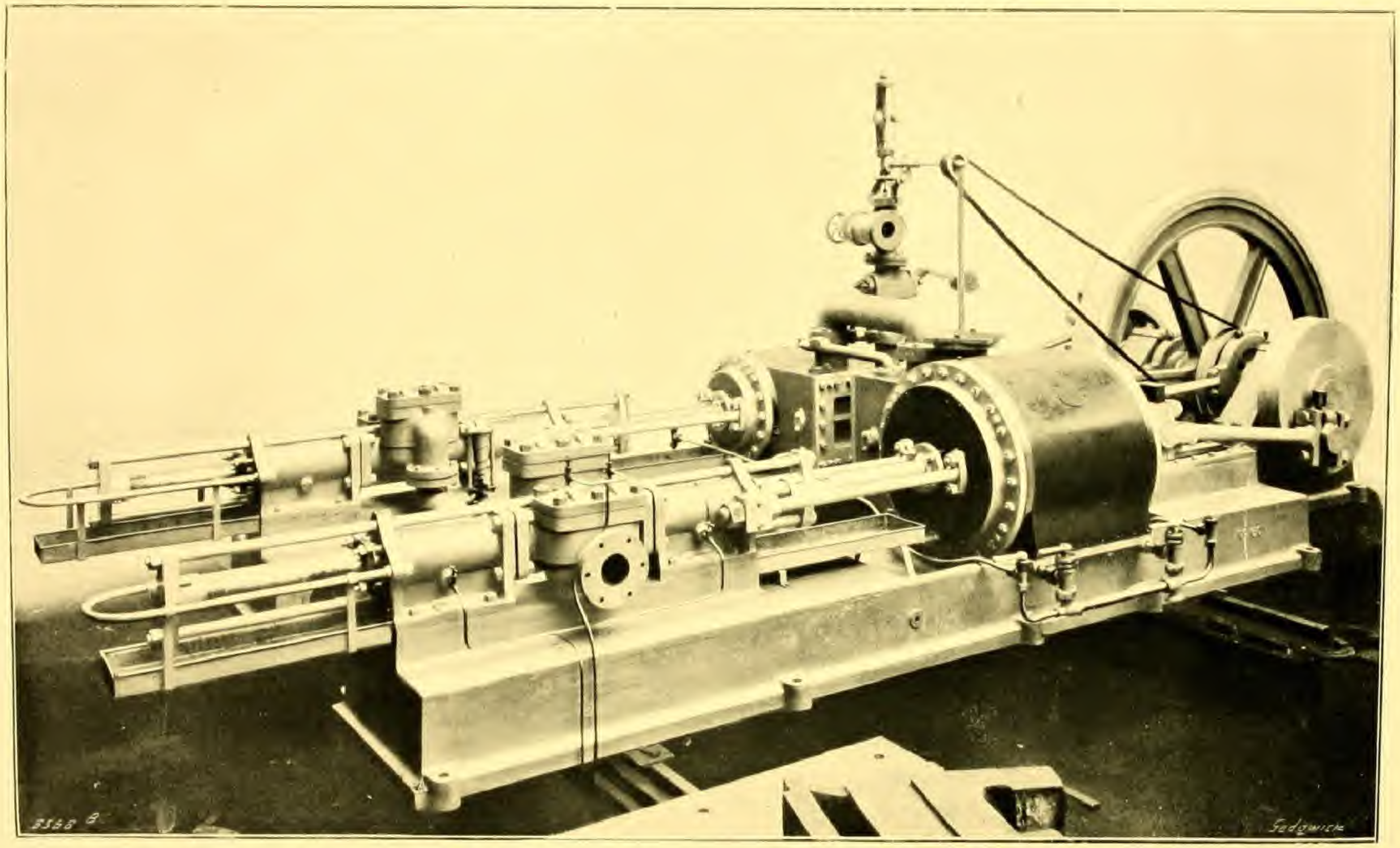
FIRST reference may be made to hydraulic pumps, in connection with which the Company have had very considerable experience. The illustration on the opposite page shows a typical set supplied for the Ordnance Works at Barrow-in-Furness of Messrs. Vickers Sons and Maxim, Limited.

The water is compressed to 1500 lb. per square inch, in four hydraulic cylinders having rams $2\frac{3}{4}$ in. in diameter. These cylinders are arranged in pairs, placed back to back; each pair is operated by one steam cylinder, and compression takes place in the water cylinders alternately. This secures a more equal turning moment, and minimises the stress on the working parts. The rams are worked direct from the piston of the steam cylinder, the ram of the rear water cylinder being connected to the steam - piston crosshead through top and bottom rods extending to the pump-rod crosshead, as shown in the engraving.

The steam engine is of the compound type, the diameter of the high-pressure cylinder being 16 in., and of the low-pressure cylinder 27 in., with a stroke of 24 in.

An extension of the piston rod through the cylinder end actuates, by disc-cranks, a shaft on which there is a fly-wheel. The normal speed is 60 revolutions per minute, which is equal to a ram speed of 240 ft. per minute.

Pumps of this type are to be found in a large number of the industrial establishments throughout the



Steam-Driven Hydraulic Pumps at the Vickers Ordnance Works at Barrow-in-Furness.

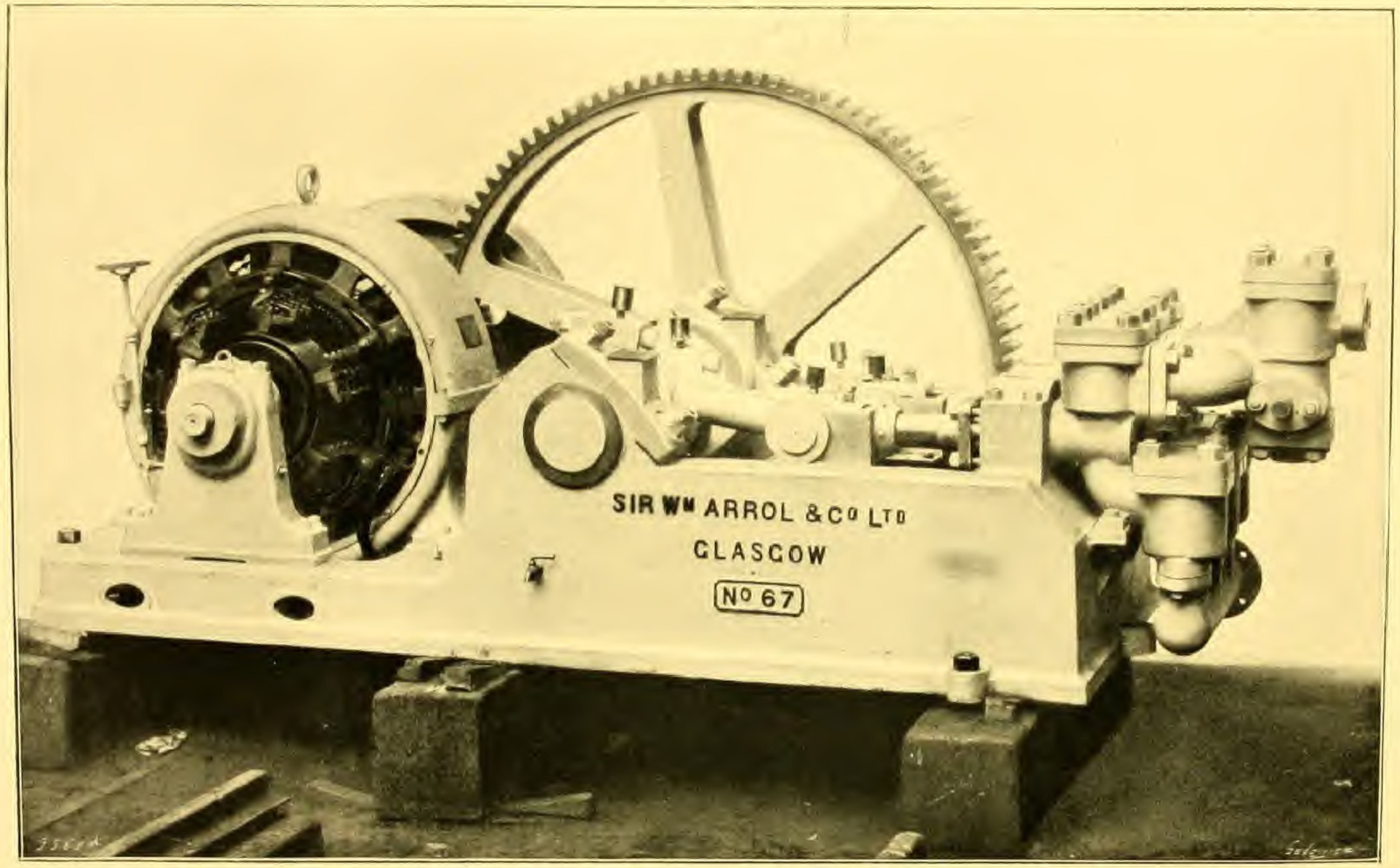
kingdom. Sir William Arrol and Company, Limited, also make pumps of corresponding power to be driven by electricity.

The view on the opposite page illustrates such a set of pumps driven by a direct-coupled slow-speed motor; and, as indicative of the general character of hydraulic pump work, we may here give a quotation from a standard specification.

"The gear-wheels are machine-cut, the pinion is in mild steel, and the wheel in tough cast iron. The crank-shaft is a steel forging of ample proportions, and the crank-pins are turned out of the solid forging. There are three single-acting pump rams working off the crank-shaft, and pumping into a common delivery pipe. The connecting-rods and crossheads are in cast steel, and the wearing surfaces are of liberal proportions. The pump castings are in mild cast steel, and the sole-plate is in good close-grained cast-iron, and is strengthened by longitudinal and transverse ribs. The pumps are bolted to the sole-plate, and rest in truly-bored seatings, with recess-checks for taking the thrust. The glands are very accessible. The pump rams, glands, valves, valve-seats, crank-shaft bushes and connecting-rod liners, are all in best gun-metal. The valves are of large diameter, and work with a very small lift, so that the wear and tear of the valves is reduced to a minimum. The motor has three bearings, and sits on an independent sole-plate, which is bolted to the pump sole-plate by fitted bolts.

"The motor will run continuously, and the accumulator controls the pumps in the following manner: When the accumulator has risen to within 1 ft. of the top of its stroke, a projecting plate engages with a ferrule on a vertical rod alongside the guides. If the accumulator continues to rise, the rod is drawn upwards, and closes a valve, to the spindle of which it is connected. By means of a small hydraulic ram at the pumps a port in the bridge pipe is uncovered, and the water pumped is passed back to the supply tank, and the motor relieved of the greater portion of its load. The pressure water in the accumulator is meanwhile held up by a check-valve at the pumps. When the accumulator falls to half its stroke, the projecting plate engages with a second ferrule on the rod, and the valve is drawn to pressure again. The small hydraulic ram at the same time closes the relief port. Pumping is resumed, and the motor again takes the whole load. A light flywheel is provided on the motor shaft to assist the motor when the load is taken off and on. The motor is controlled by a simple starting switch, and the current consumed when running without load is small."

If the service required of the pumps is intermittent, a controlling switch is arranged which is worked by the action of the accumulator, in a similar manner to that described.



Electrically-Driven Hydraulic Pumps.

Hydraulic Cranes.

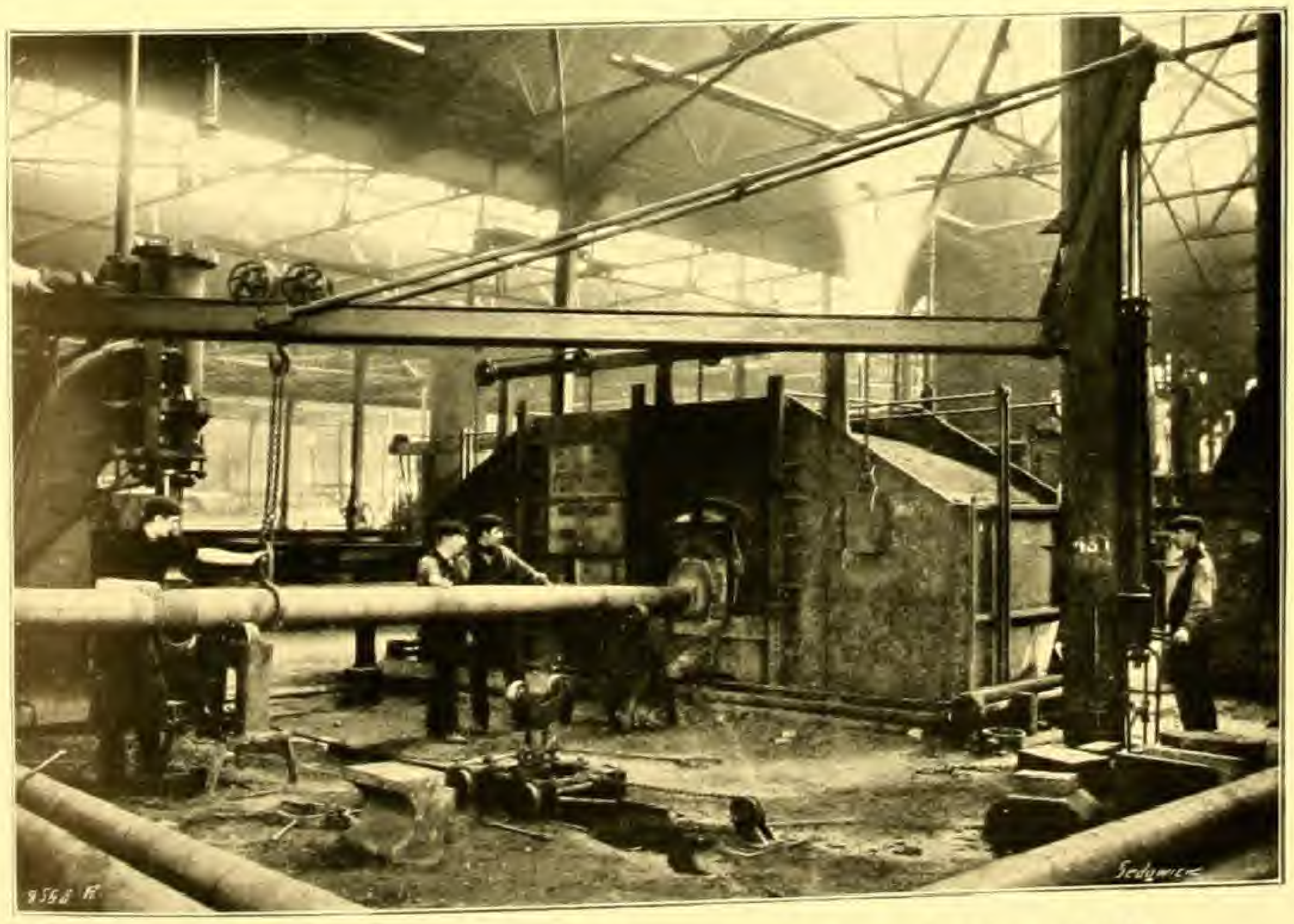
IN bridge and constructional steel work generally, it frequently occurs that difficulties are involved in the arrangement of manufacturing and erecting plant, because of limitations in the space available, and few firms have had to face more of such problems than Sir William Arrol and Company, Limited.

This resulted in the development of an important department at the Dalmarnock Works at Glasgow, for the manufacture of cranes to meet almost every contingency. Now few shipyards are without some of Sir William Arrol and Company's hand, hydraulic, or electric jib-cranes for feeding machine tools and for other purposes, and several of these are illustrated and described on the five succeeding pages.

The first illustration shows an hydraulic jib-crane, with racking and slewing motion. It is made in various sizes up to 30-ft. radius. The jib is at a fixed height, and the hook block rises and falls through a height of 12 ft. to 20 ft. The lifting power ranges from 5 to 20 tons.

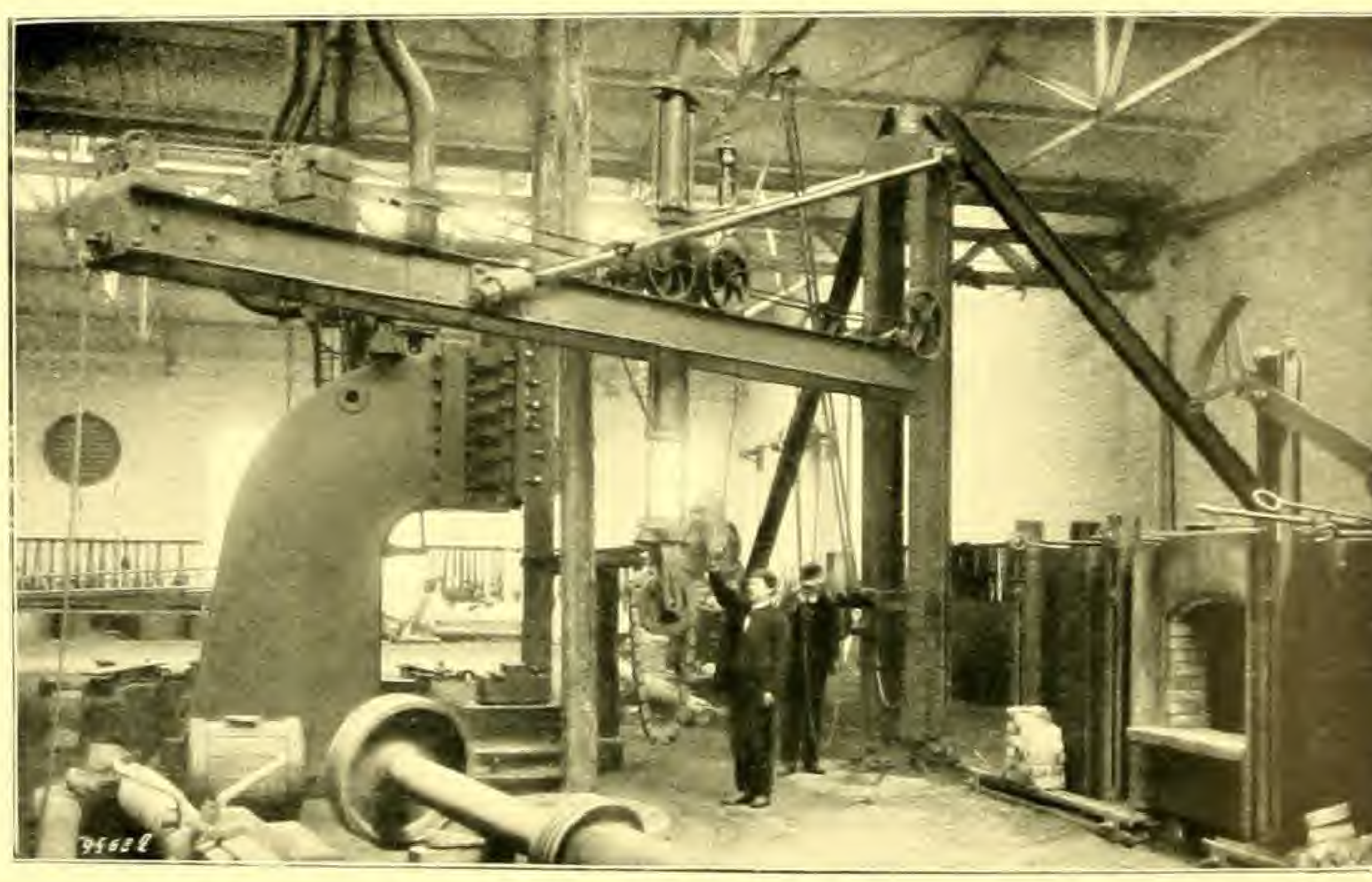
The second illustration shows a type of crane largely used for supporting hydraulic riveters and other work. The crane occupies little floor space, and has also the advantages of facility, speed of movement, and cheapness. This type is manufactured of various radii from 20 ft. to 40 ft., with an hydraulic ram capable of lifting a maximum load of 5 tons through from 5 ft. to 12 ft.

Hydraulic Jib Crane, with Racking and Slewing Motion, at the North British Locomotive Works, Springburn.



Crane for Supporting Hydraulic Riveters and other Work.

On this and the opposite pages there are views of two types of crane, both at the Naval Construction Works at Dalmuir of Messrs. William Beardmore and Co., Ltd. These illustrate modifications of standard types to suit special requirements. The crane shown below is in the forge; the other is in the boiler shop. The design of each is clearly indicated. The speed of lifting of these



Forge Crane.

cranes is kept low, varying from 5 ft. to 10 ft. per minute, as the range is slight and the work must be kept under control.

Several types of crane have been designed for dealing with plates and angles in shipbuilding and bridge building yards. A typical crane, illustrated on page 220, is one which lifts a load of 3 tons at a radius of 40 ft. At 20 ft. radius the crane is self-balancing. The foundations for the mast are at 7 ft. below the ground level, and there the load is taken on hard steel and gun-metal washers, with

Light Service Boiler Shop Crane of 4-Tons Capacity at the Naval Construction Works of Messrs. William Beardmore and Company, Limited

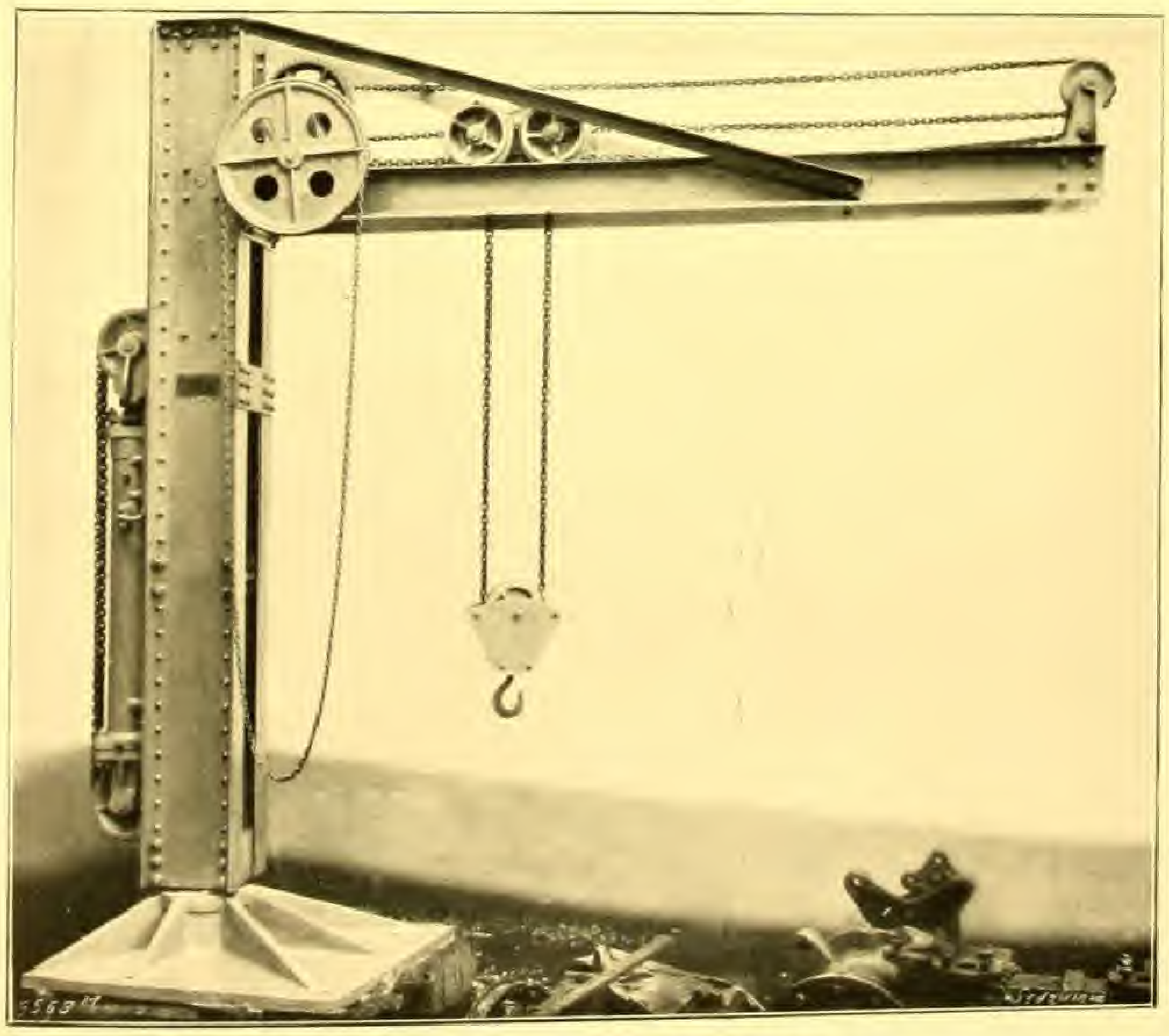
efficient lubrication. The central bearing is at the ground level, and consists of a roller path of large diameter, the external circumference of which forms the slewing drum. The slewing cylinder is fixed to the side of the mast, and the lifting cylinder is within the girders forming the mast. The jib is built up of channels and angle-ties securely braced together. The crane is built to swing through a

A Typical Shipyard Crane at Messrs. John Brown and Company's Clydebank Works.

complete circle, which is a great advantage. A crane of this type is illustrated on the view on this page.

The first of the illustrations on the opposite page shows a 3-ton self-contained hydraulic crane, standing upon its own pedestal, as constructed for Messrs. Barclay, Curle and Co.'s shipyard. These cranes are very useful in shipbuilding yards for handling plates and angles at the racks. They are made of moderate capacity, up to 5 ton and 30 ft. radius.

3-Ton Pedestal Crane at Messrs. Barclay, Curle and Company's Shipyard.



Crane at Vickers Works at Barrow-in-Furness

Another form of self-supporting crane is shown in the second illustration on the preceding page. In this case the mast swivels round a central forged-steel pin, which is shrunk into a cast-iron base-plate. The load can thus be brought much nearer the mast than with a pedestal crane. The hydraulic mechanism is placed in the rear of the mast, and assists in balancing the crane.

Hydraulic Riveting Machines.

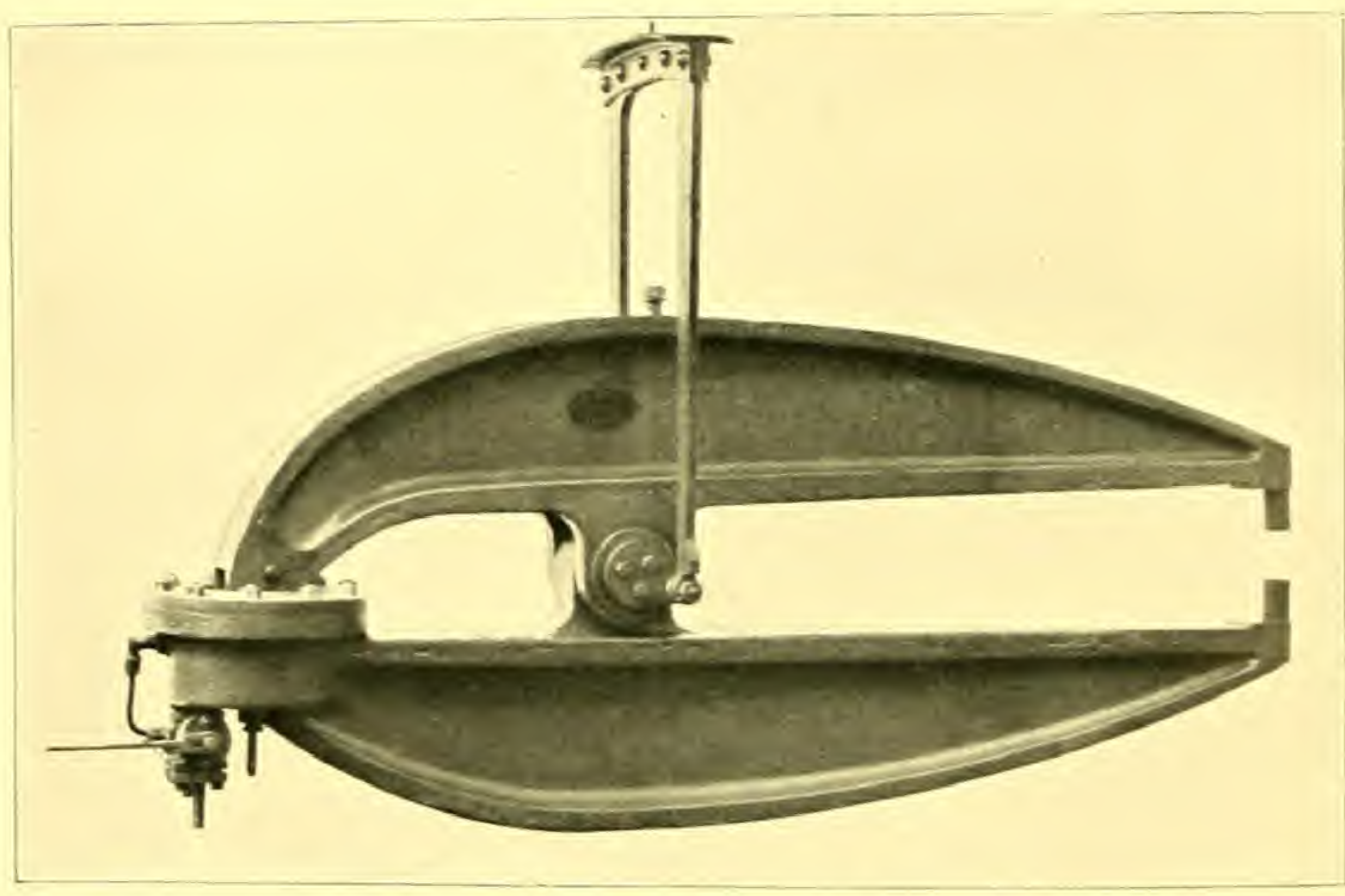
AS we have already indicated, Sir William Arrol was the first to apply hydraulic riveting to bridge structures, and the system he then evolved was at once applied in connection with the building of large ships and constructional work generally. From time to time experience has enabled improvements to be effected and new forms to be evolved to meet unusual conditions. Thus, tools are now manufactured to suit all possible requirements.

Silence in working, freedom from vibration, and the minimum of wear and tear are among the advantages claimed for these machines. The pressure exerted on the rivet is gradual, and can be maintained until the rivet is cold, which, in the case of thick plates, is important. In girder work about 200 rivets per hour can be closed with this make of riveter.

On the six succeeding pages there are illustrated and described several types of portable riveters, used largely in connection with the riveting of heavy girders, of the double-bottom structure of large steamships, and of a great variety of other structures.

"Scissors" riveters (as illustrated on the next page) are made up to 75 in. gap. The hobby arms are cast in Siemens steel, and truly machined on large side-bearing surfaces at the hinge; the hinge pin is of the best forged steel, and the dies are of hard cast steel. The cylinder cover is of mild steel, the piston of gun metal, and the

working valve, of the piston type, is of gun metal. A relief arrangement is provided, so that, in the event of the arms being closed when the dies are not in place, no damage is done in the cylinder head, the water escaping. Two hangers are provided, so that the dies may be worked in a horizontal or vertical position; by moving the point

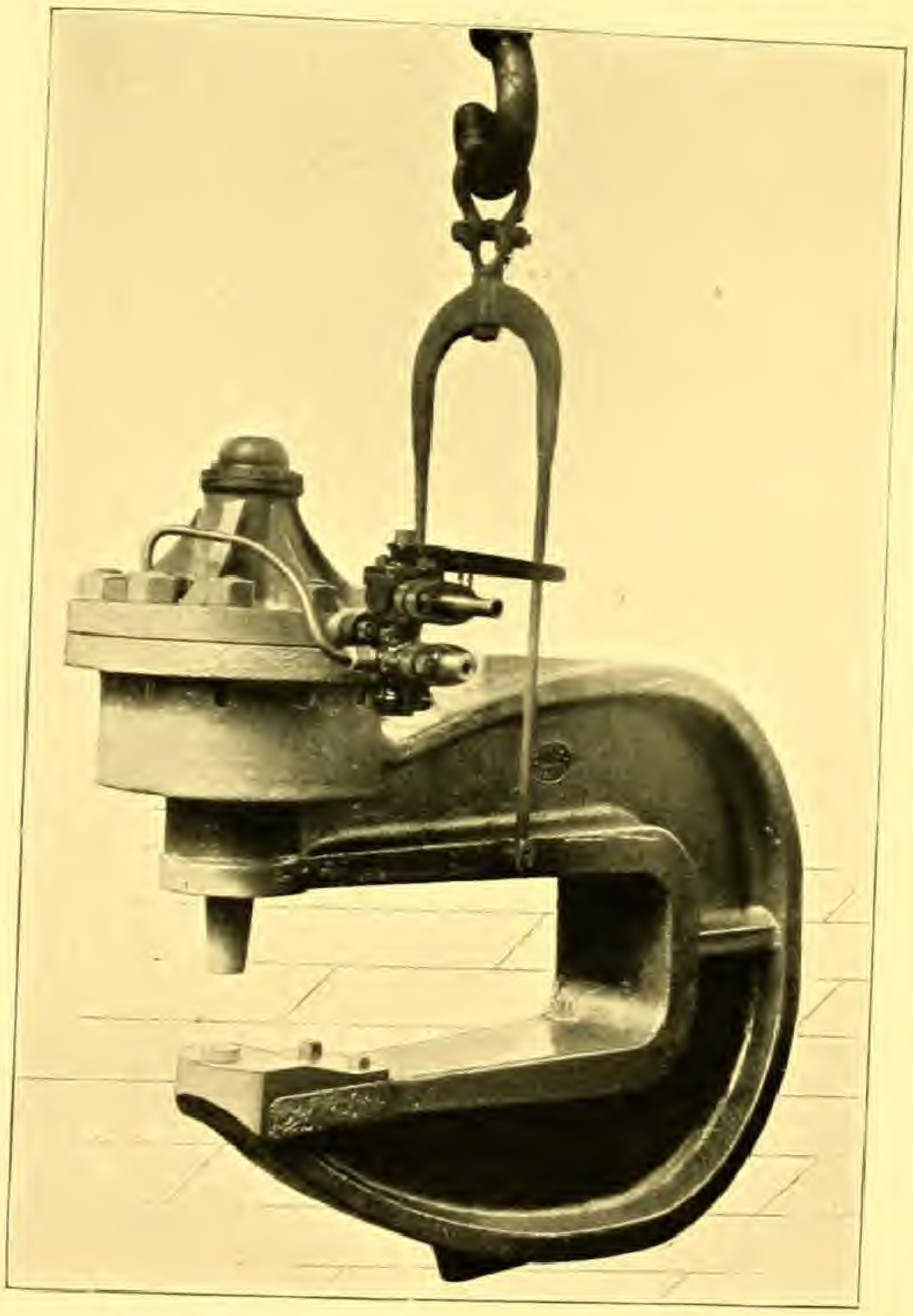


The "Scissors" Type of Hydraulic Riveter.

of the suspension to the right or left along the hanger the dies will lie at a corresponding angle. The machine is specially suitable for riveting in corners or confined spaces.

The "bow" riveters, of which a type is illustrated on the opposite page, range from 24 in. to 69 in. gap. The main casting is in Siemens Martin steel; the piston is a steel forging, and the cylinder is lined with gun metal. The operating valve is of the piston type, and is in gun metal.

The machine illustrated on this page is shown suspended to a link hanger, but in others a projecting web is carried round the back of the outer flange, and is bored with

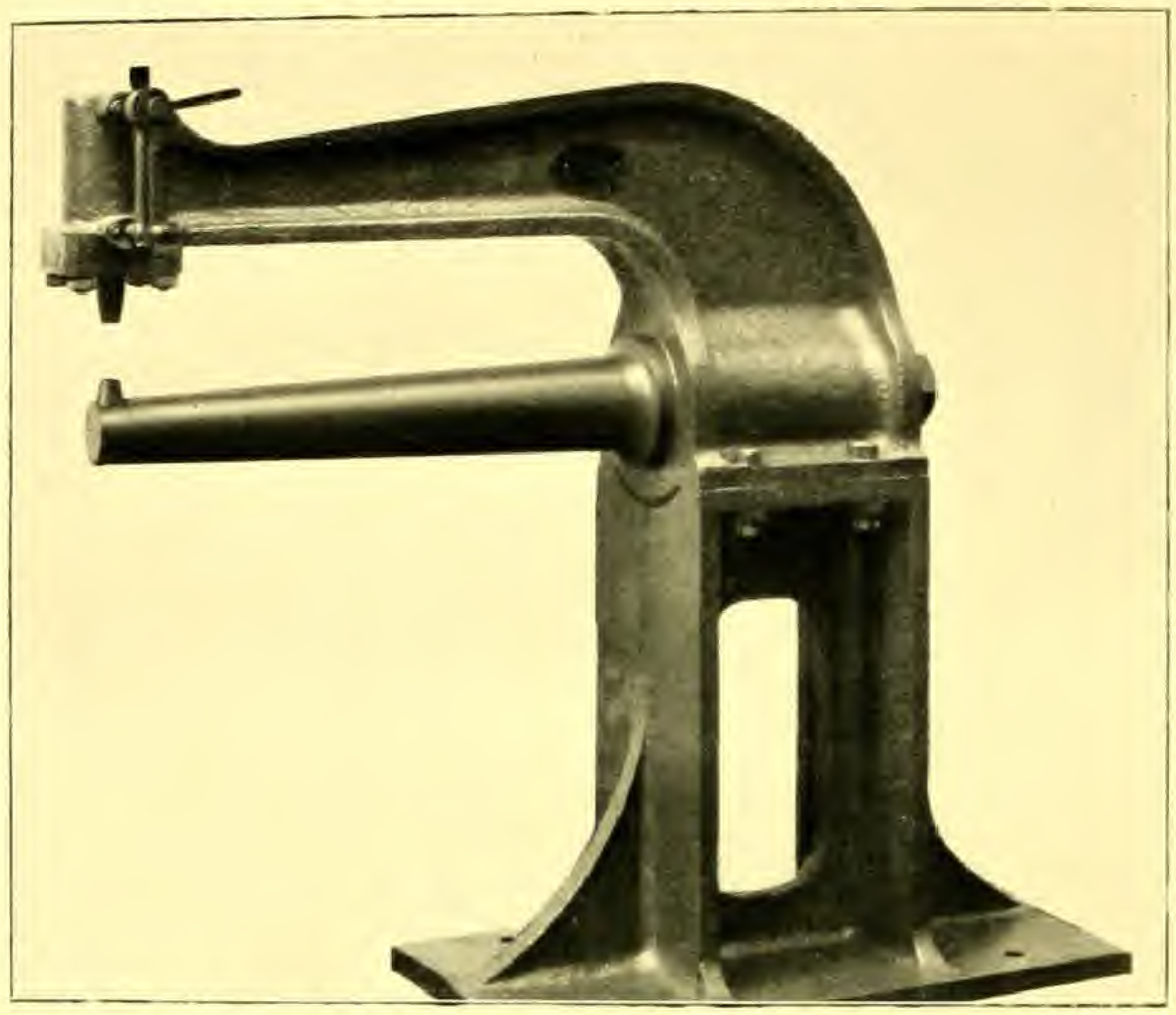


The "Bow" Type of Hydraulic Riveter.

bolt holes, closely pitched, so that the machine can be slung with the riveting dies either in a horizontal or in a vertical position.

By the engraving on the next page there is illustrated a machine for riveting light steel-plate pipes from about 5 in. in diameter upwards. For this work the cylinder

is, as a rule, proportioned to give a pressure of 5 tons on $\frac{3}{8}$-in. rivets. The cylinder is lined with gun metal, and

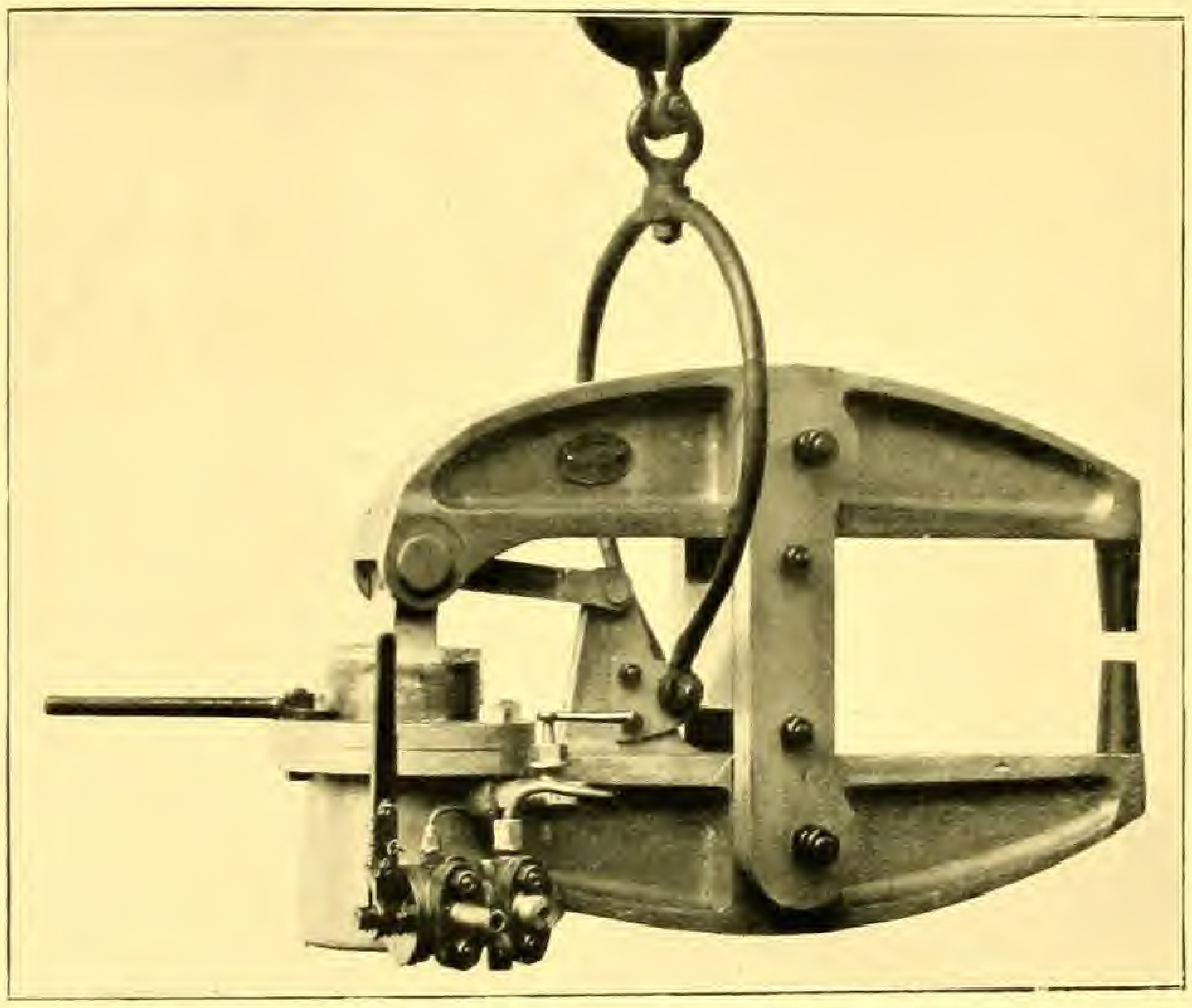


Machine for Riveting Light Steel Pipes.

is formed in the frame of the machine, which is a steel casting. The holder-up, as shown in the engraving, is circular in form. The use of high-tensile steel minimises its recoil. The holder fits tightly into a socket in the main castings. The machine is mounted on a cast-iron stand, and has proved very convenient in connection with the making of the light steel pipes now so greatly in demand for water and other supply-mains.

The "hinged" type of riveter, which forms the subject of the next engraving, has a patented parallel motion, by means of which the pressure is exerted in the line of the rivet, so that any danger of twisting the rivet head is obviated. This riveter is very largely used,

as it is light and handy, and suitable for a great variety of work.



"Hinged" Type of Hydraulic Riveter.

The engraving on page 229 illustrates the application of hydraulic riveters. The view shows the double-bottom, up to the margin plate, of one of the recently built turbine-driven Cunard liners.[1] Many of the plates were $1\frac{1}{8}$ in. thick, 32 ft. long, and $5\frac{1}{2}$ ft. in width. In many cases the plates were quadruple riveted; the sheer strake was quintuple riveted, $1\frac{1}{8}$-in. rivets being used.

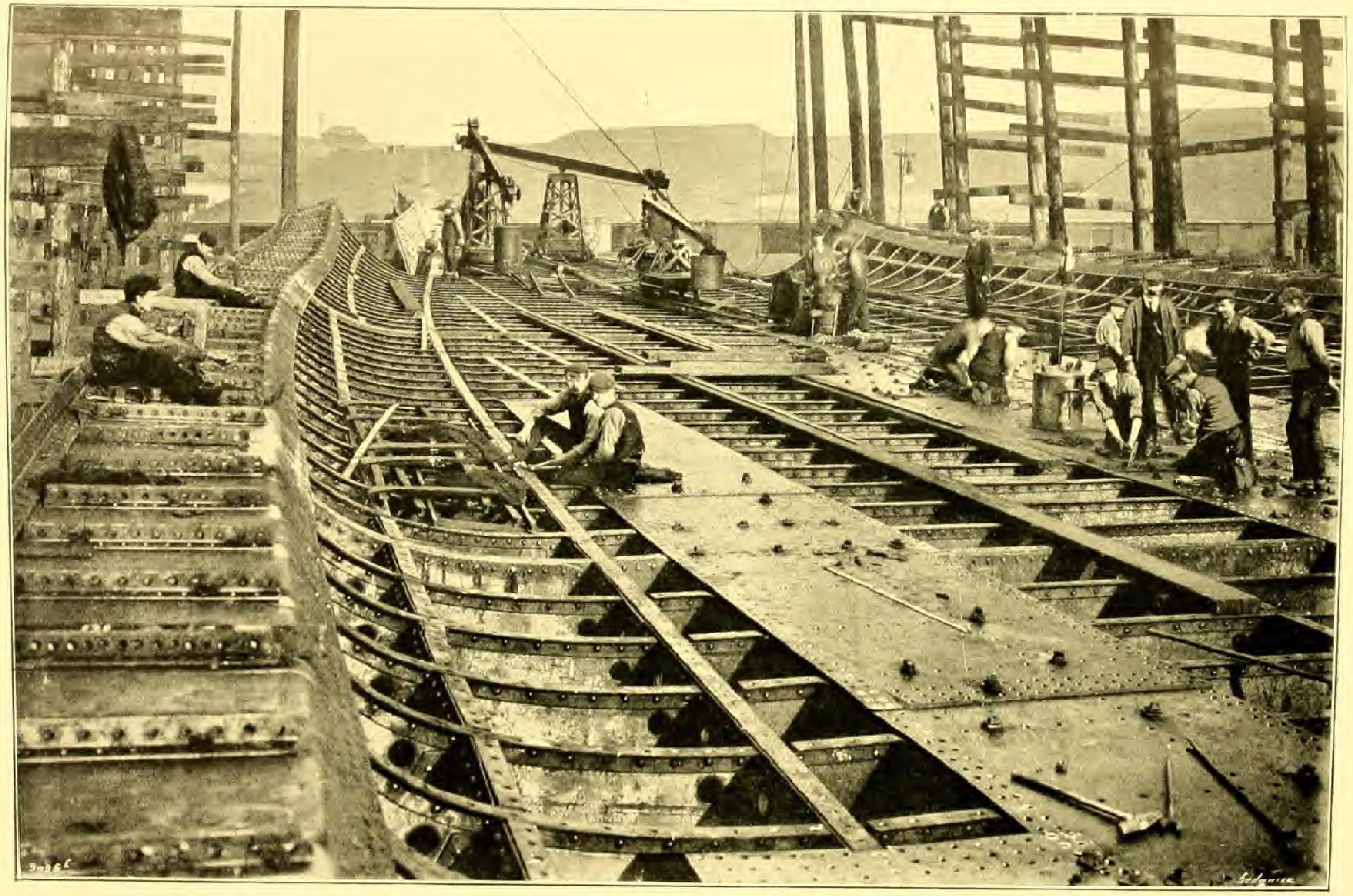
The riveting done by hydraulic power included the centre girder keel-plate, garboard strake, the centre of the inner bottom, the intercostal girders, the end frames to the reverse angles of the beam-knee brackets, the bridge

[1] See ENGINEERING, vol. lxxx., page 716; vol. lxxxi., page 729.

deck sheer strake, the shelter deck stringer angles, and the side stringers between the web frames.

The rivets in the shell and tank top plating vary from $\frac{3}{4}$ in. to $1\frac{1}{4}$ in. in diameter, spaced on an average four to five diameters apart. In the bulkheads the rivets are generally $\frac{3}{4}$ in. in diameter, spaced four to five diameters apart; the deck rivets are $\frac{3}{4}$ in., spaced four to five diameters apart.

The riveting machine was carried on a beam which had on the opposite end a counterbalancing weight. This beam was supported in the centre upon a lattice-work column running on wheels, or on a small truck on the railway track laid on each successive deck as soon as there were beams to carry it.



Riveting a Cunard Liner at the Clydebank Works of Messrs. John Brown and Company, Limited.

Hydraulic Retort-Machinery for Gas Works.[1]

RESEARCH work in general hydraulic mechanism led Sir William Arrol, early in the 'nineties, to associate himself with the late Mr. William Foulis, the gas engineer to the Corporation of Glasgow, in experiments which resulted in the evolution of hydraulic machinery for undertaking the most laborious work in connection with the production of coal gas.

This plant now includes a machine for charging the retorts, another for withdrawing the coke, and a third for cleaning the ascension pipes. The labour cost is greatly decreased, the time occupied in the various operations has been minimised, and the output of gas from a given number of retorts has been increased.

The first commercial application of the system was in 1894. Since then, nearly all the large towns in Great Britain, and many abroad, have adopted the plant. There are installations at Vienna, Amsterdam, Berlin, and Cleveland in the State of Ohio. In London the Gas-Light and Coke Company, the South Metropolitan Gas Company, the Commercial Gas Company, and others use the system extensively. It is also applied at works in Bromley and

[1] See Professor Jamieson's "Applied Mechanics," vol. ii., page 324 (Griffin); Proceedings of Institution of Mechanical Engineers, 1895, Parts III. and IV., page 331; ENGINEERING, vol. lx., pages 153, 312; vol. lxxxi., page 415.

Southend. In the provinces it is only necessary to mention Birmingham, Glasgow, Liverpool, Leeds, Hull, Bolton, Brighton and Dundee, to indicate the wide extent of the utilisation of the system.

There are certain variations in the mechanism to meet different conditions, but the general principle is the same; and it is, perhaps, only necessary to describe one installation in detail.

The carriage for all three machines is of the same general construction, and the same method of traversing is adopted in all cases. The frame is a simple rectangular structure of plates and angles, carried on axle brackets of cast steel, and of dimensions to suit the height and width of the retort-charging platform. The machine is traversed along the front of the battery of retorts by an hydraulic motor, which works through bevel gearing direct on to the axle.

The size and power of the motor varies with the different machines. Thus, the charging machine, which has a weight of about 13 tons, including 5 tons of coal in the hopper, is traversed by a 4-horse-power motor at a speed of 70 ft. per minute; the drawing machine, of about 4 tons, is traversed by a $1\frac{1}{2}$-horse-power motor, at a speed of 150 ft. per minute; and the ascension pipe-cleaning machine, of 6 tons, has a $1\frac{1}{2}$ brake horse-power motor of the capstan type, and is traversed at a speed of 100 ft. per minute.

In the frame thus traversed there is carried a beam built up of steel channels, to support the gear for charging, for withdrawing, or for cleaning the pipes. In order that the level of this platform may be varied to suit the height of the mouthpiece of the retort, there is hydraulic mechanism for raising and lowering it, as clearly

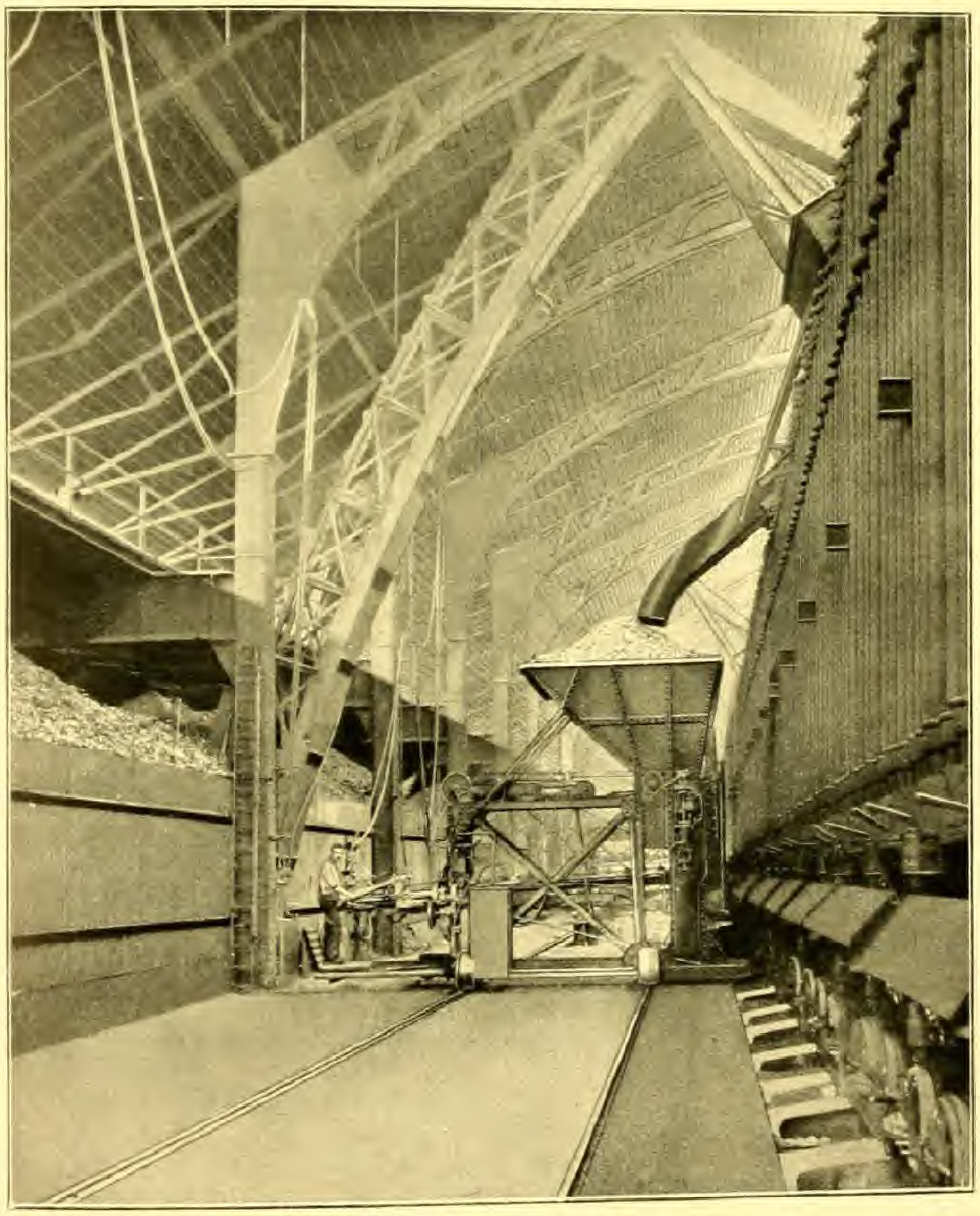
shown in some of the views. The steel channels of this horizontal beam or platform work in vertical angle guides on the main framing, and the whole is suspended on two lifting chains, one at the front and the other at the rear. These chains pass over guide pulleys carried on the top of the framing, and thence over separate pulleys secured to the ram-head of the hydraulic mechanism. The position of this elevating cylinder varies in different machines. In some cases it is on the top of the frame, working horizontally; in other instances it is placed vertically on the side of the frame, but the action is the same.

Directing attention now to the mechanism for charging retorts, a word may first be said regarding the supply of coal. This varies with the general arrangement of the retort-house.

At the Beckton Works of the Gas Light and Coke Company, Ltd., as illustrated on the opposite page, the coal wagons are discharged into coal stores under the track at the side of the retort-house, whence the fuel is conveyed by elevators to hoppers built over the retorts. The coal is fed from these through shoots to a hopper on the charging machine, as shown clearly on the engraving opposite. For regulating the supply there is fitted to the shoot a valve, which is manipulated from the retort floor. The total height of the Beckton retort-house from ground level is 55 ft., and the width 70 ft. In the various retort-houses there are sixteen charging and sixteen drawing machines, with several ascension pipe-cleaning machines.

The coal drops from the hopper on to the charging pan through a regulating drum, built of a series of plates radiating from the centre. This drum, fitted at the base of the hopper, is rotated by an hydraulic cylinder. It is

thus easy to vary the rate of flow of coal according to its quality. The coal drops on to the shoot pan, which bridges the distance between the machine and the mouth



Retort-Charging Machine at the Beckton Works of the Gas Light and Coke Company, Limited.

of the retort; and as it falls on to this pan it is pushed into the retort by a shovel actuated in successive strokes by an hydraulic ram on the horizontal beam.

This hydraulic ram, like all others in the system, works at from 400 lb. to 700 lb. pressure, which experience has shown to be the most suitable, as it minimises the diameter of the cylinders and flexible piping. The latter is of india-rubber and canvas, bound with wire, and tested to 1500 lb. per square inch.

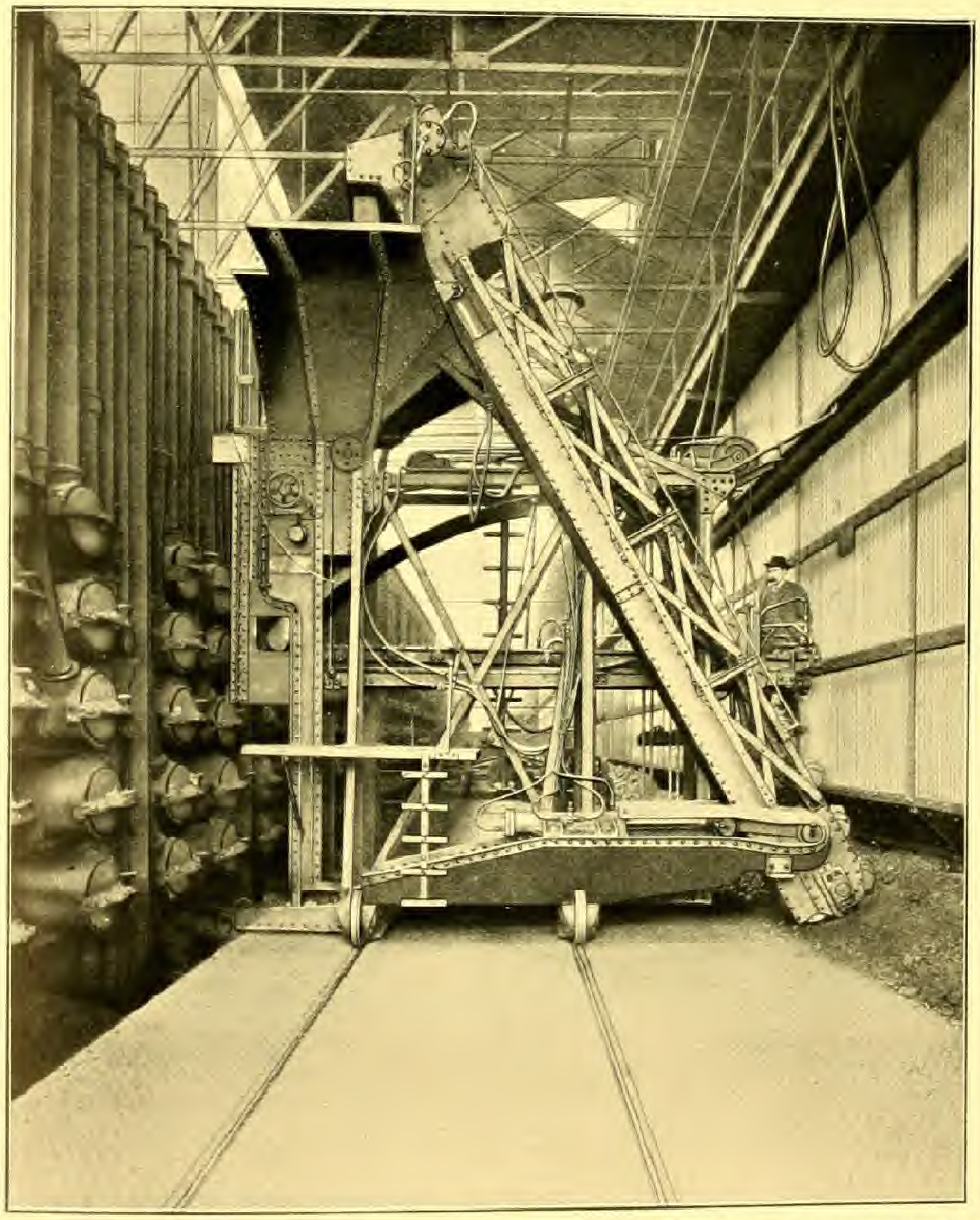
The coal is, at the first stroke of the ram, pushed to a point about 12 in. from the back of the retort—or from the centre in the case of a double-ended retort—and successive charges are then deposited about 18 in. apart; the depth of coal in the retort being regulated at about 8 in. The distance apart of successive charges is fixed by a bar on the beam which actuate stopper-plates to engage with the charging ram at each successive stroke. Thus, when the machine has once been set, the whole operation is automatic.

The machine is completely controlled in all its connections from one platform, the handles for operating all the valves being centralised there, and a retort, 20 ft. long, can be charged from both sides, as at the Beckton Works, in from 15 to 20 seconds. The average time occupied, including the work of traversing the machine from retort to retort, and setting it to suit the height, is about 45 seconds, and as many as forty-eight retorts have been charged per hour, allowing sufficient time each hour to give the men operating the machines a rest.

Where the coal is of a quality to give off its gas in three hours, one machine would suffice for 120 retorts; but where the coal requires six hours for generating gas, each machine could deal with 240 retorts.

There are several alternative arrangements for dealing with the coal. An elevator is sometimes fitted to the

machine for picking up coal from the side bin, and discharging it into the hopper at the top. This machine, which is illustrated on this page, is very useful in retort-houses



Charging Machine at the Vauxhall Works of the South Metropolitan Gas Company, Limited.

with side coal-stores. The machine illustrated is in use at the Vauxhall Gas Works of the South Metropolitan Gas Company, Ltd. The elevator buckets work freely in the

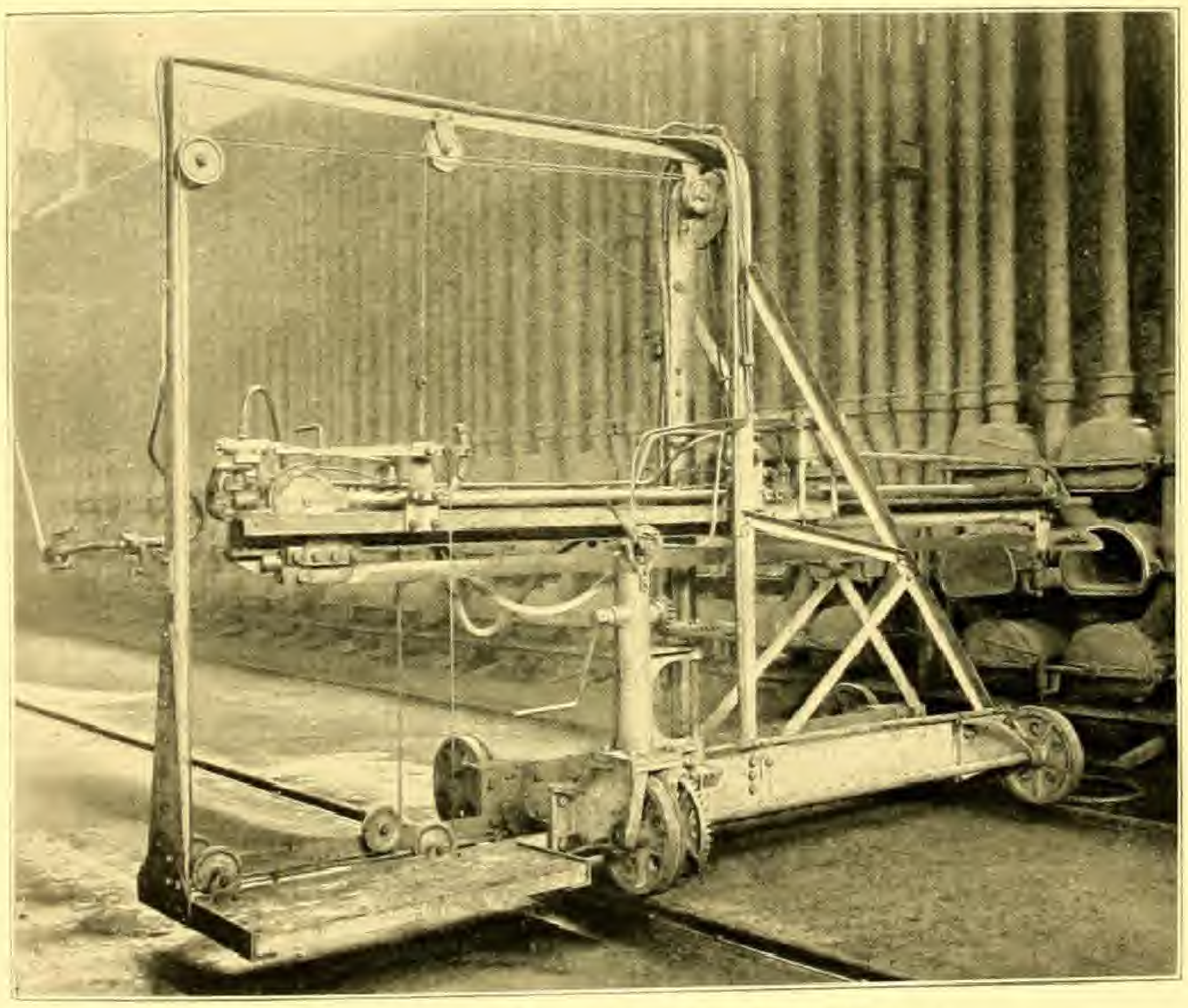
projecting skirt of coal, and are drawn back out of the coal when the machine is travelling. For this purpose, an hydraulic cylinder is fitted to the bottom end of the elevator frame. The top end of this is pivoted, while the bottom end is mounted on side rollers running in guides at the side of the machine. These guides have a slight upward gradient, so that the bottom of the conveyor is raised clear of the coal on the floor. All the other motions are as on the ordinary machine, and the operator controls all motions, including the manipulating of the elevator.

The drawing machine is simpler in construction. As in the case of the charging machine, there is a horizontal beam carrying two hydraulic cylinders, one for driving a rake forward into the retort, and the other for drawing it out. The lever which actuates the valve for working the rake is so attached to the beam that the movement which sets the hydraulic valve for the forward movement of the rake simultaneously depresses the back end of the beam, raising the rake to allow it to pass over the coke in the retort. The reverse movement of the lever, while actuating the valve of the cylinder for withdrawing the rake, elevates the back end of the beam, whereby the end of the rake is lowered into the coke.

The rake does not withdraw the full contents of the retort at one stroke. At each successive stroke it enters the retort for a greater distance, the length of the travel being regulated at the discretion of the operator by the extent of the opening of the hydraulic valve. The return stroke of the rake is checked by an india-rubber buffer, placed at the end of the horizontal beam; but a little experience enables the operator to regulate the motion by the working of the valve.

The greater part of the exhaust water is returned to a cast-iron box, placed above the beam, whence it is allowed to flow continually over the rake-head and rod to cool them.

Various forms of rake-heads have been tried. The form now applied consists of a cast-iron head pivoted to the rake to enable it to swing slightly.



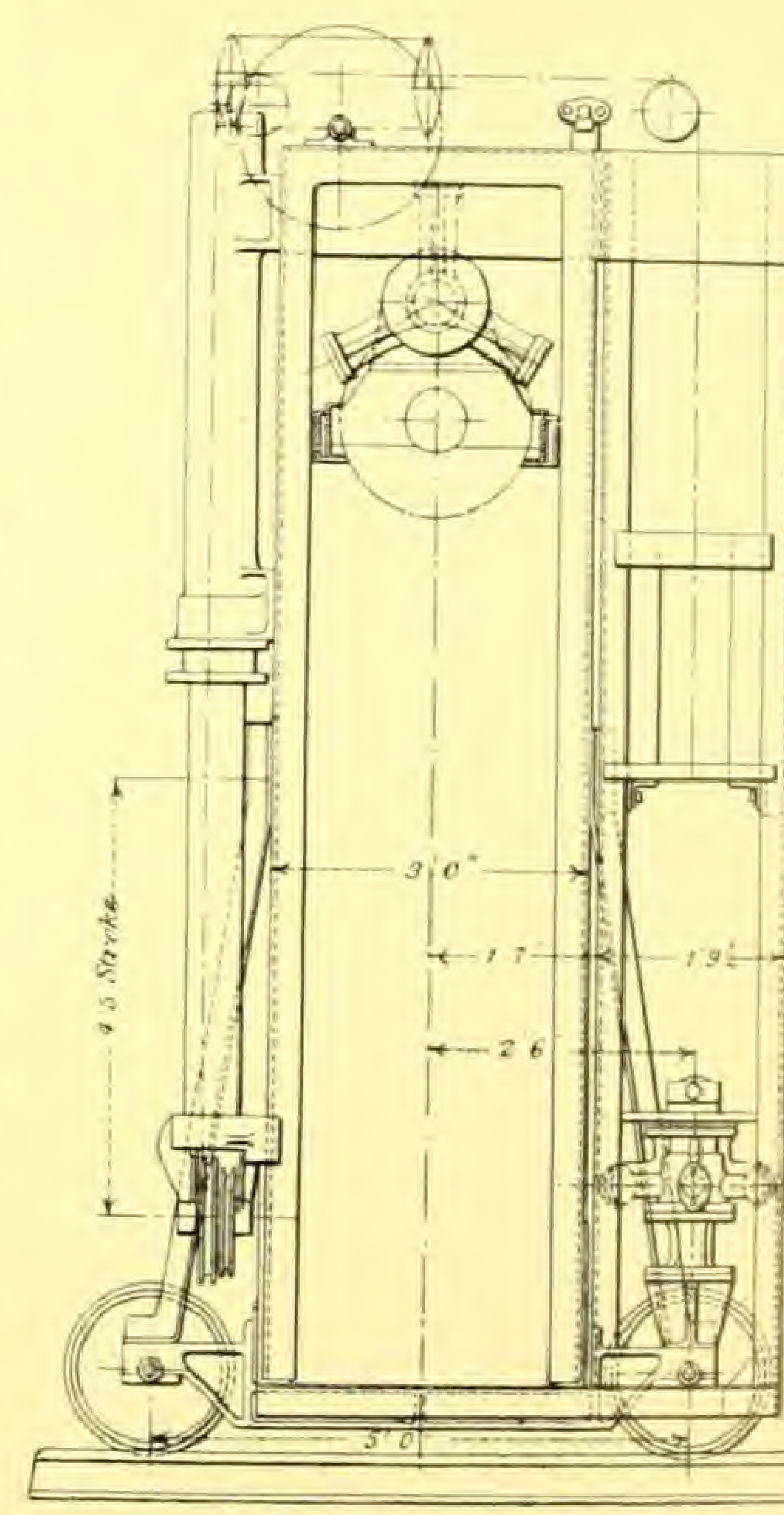
Machine for Withdrawing Coke from Retorts.

This machine takes about thirty seconds to draw a charge from the retort.

The latest development is in the construction of a machine on the same lines for cleaning the ascension-pipes of retorts. Many attempts have been made in the past to construct such a machine, but none succeeded until Mr. Andrew S. Biggart, of Sir William Arrol and Company, Limited, in collaboration with Mr. G. C. Trewby, the late

chief engineer of the Gas Light and Coke Company, Ltd., evolved a method whereby a flexible shaft with an auger point, is forced up the ascension-pipe by hydraulic power, while being rotated at a speed to suit the density of the deposit on the pipe.

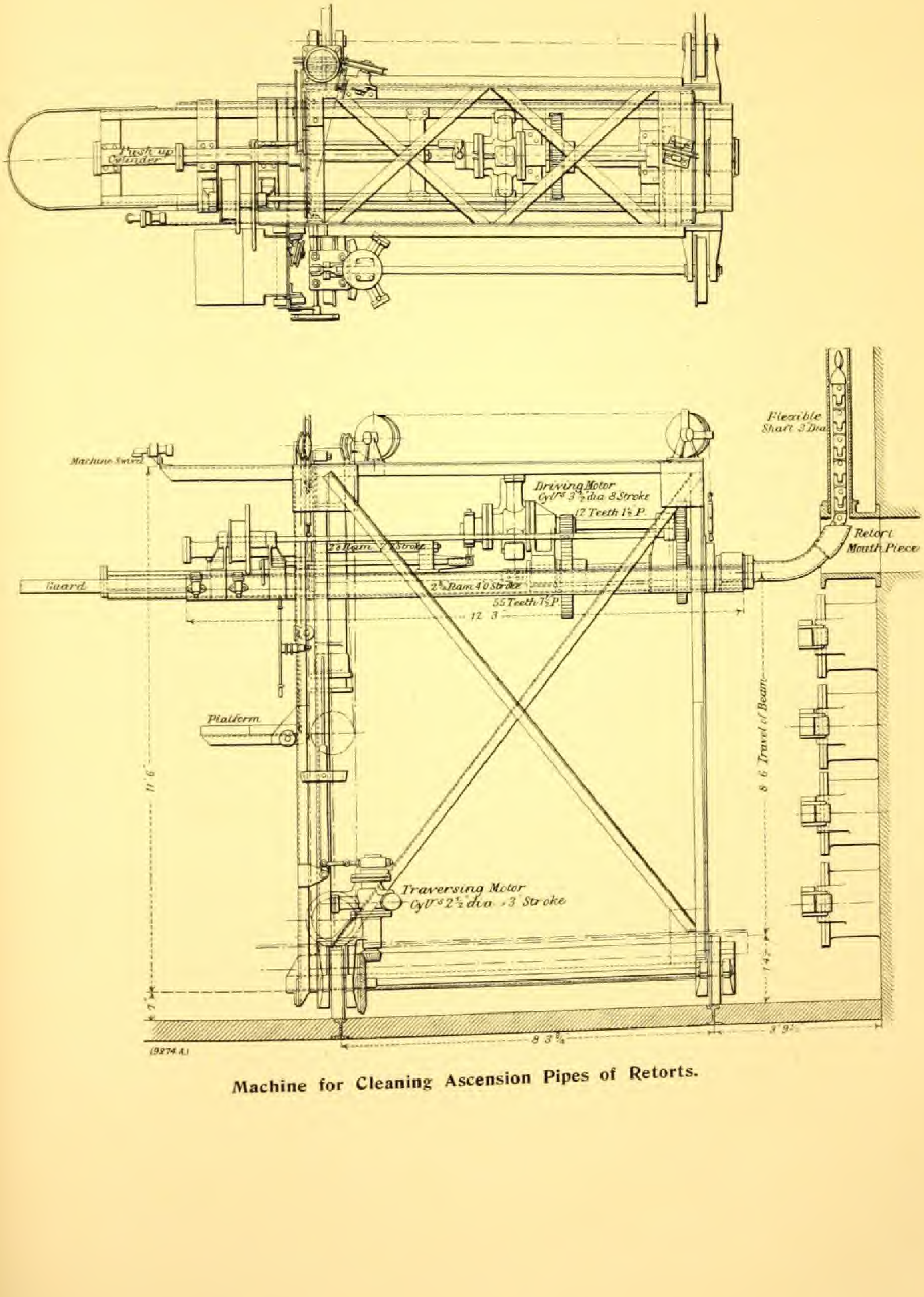
This machine was first tried at the Beckton Works of the Gas Light and Coke Company in 1905.[1] There were some difficulties incidental to the first application, but from the outset it was evident that the inventors had worked on the right lines; and now, as the result of experience, various improvements have been made, some of them suggested by the engineering staffs of the Gas Light and Coke Company's Works, and of the Vauxhall Works of the South Metropolitan Gas Company. These improvements have been embodied in the later machines, of which illustrations are given in pages 239 and 241.



Machine for Cleaning Ascension Pipes of Retorts.

The frame and traversing mechanism are the same as in the charging and drawing machine. The beam carrying the hydraulic mechanism for cleaning the pipe is similarly raised and lowered within the frame. At the outer end of this beam there is fixed an hydraulic ram $2\frac{1}{8}$ in.

[1] See ENGINEERING, vol. lxxxi., page 415.



(9274 A.)

Machine for Cleaning Ascension Pipes of Retorts.

in diameter and 2 ft. 2 in. stroke, for racking the cleaning shaft inwards and outwards. The change-speed wheels, to be referred to later, and the motor for rotating the shaft for cleaning the pipe, are supported on brackets, which assist to bind the framework together, and give it stiffness.

The design of the shaft for cleaning the pipe is the result of considerable experiment. Toothed ferrules strung on a wire rope, with ball-and-socket joints to take the thrust, afforded considerable success; but subsequently it was decided to build up the shaft of a number of sections, as this construction readily adapted itself to inequalities in the pipe. Experience also showed that a variation in the speed of the shaft became necessary, as the tarry deposit was at times hard and tenacious, at other times stiff and glueish, or soft and easily removable. The maximum efficiency could not be got always with a uniform speed. It was therefore decided to arrange differential speed gear in connection with the rotation of the shaft.

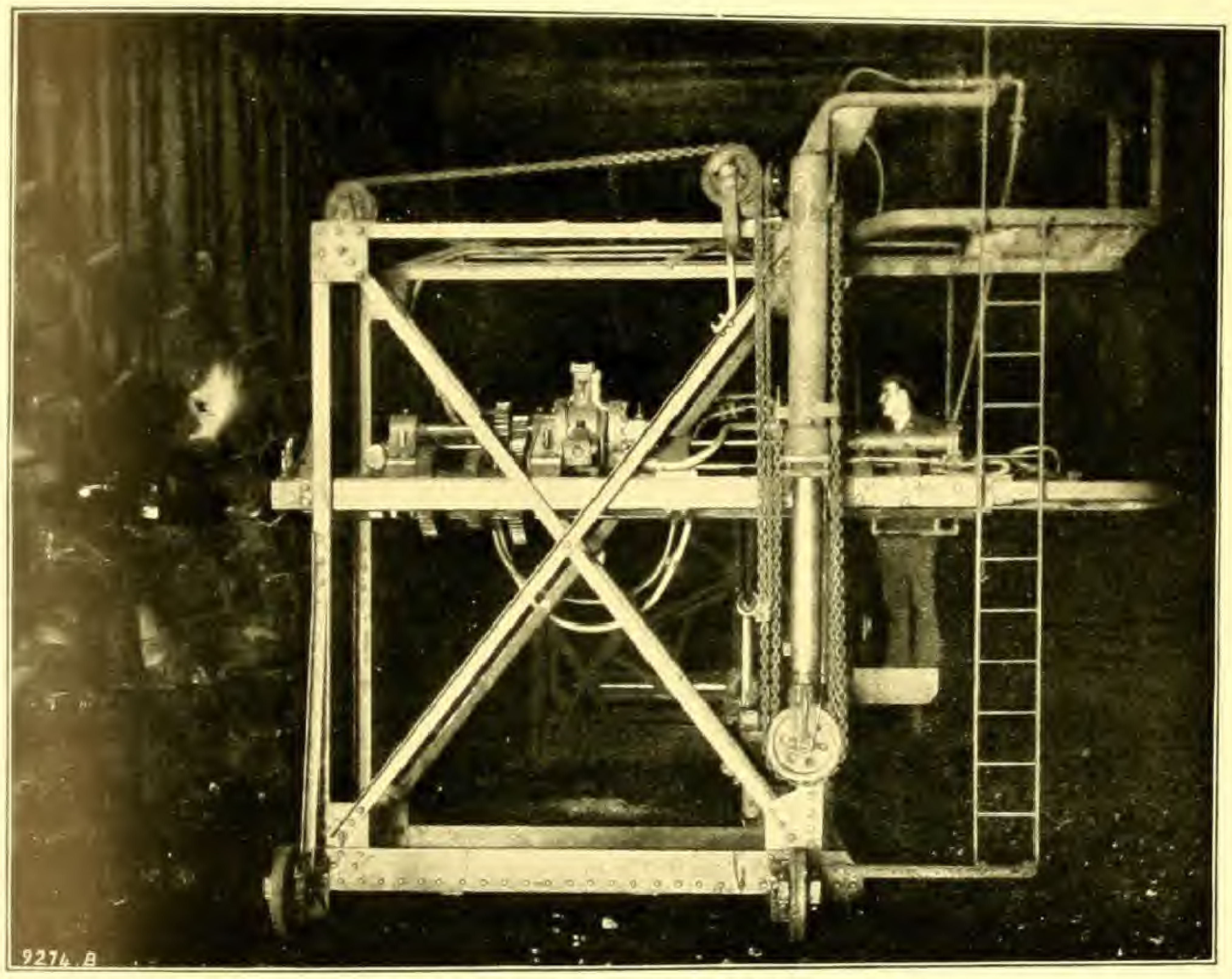
To admit of lateral motion for the driving of the shaft up the pipe, the wheels for rotating the shaft are mounted on a sleeve, into which the shaft is keyed by a sliding feather. The shaft is driven forward through the sleeve and up the ascension pipe by means of a horizontal hydraulic cylinder, while the sleeve and shaft are rotated by a triple-cylinder hydraulic motor working a countershaft, on which are pinions gearing into the spur-wheels on the sleeve.

All the motions are independent of each other, and are controlled by separate cocks; the whole of the working handles are located together, convenient to the operator standing on the end platform.

The cleaning machine in ordinary work at the Vauxhall Gas Works satisfactorily deals with fifty-two pipes

within an hour, allowing sufficient time for rest for the operator upon the completion of each lot of pipes.

The practice, where charging, withdrawing, and cleaning machines are used, is for these machines to follow each



Machine for Cleaning Ascension Pipes of Retorts.

other in the order desired. All of the pipes are systematically cleaned each shift, although this may not be necessary in all cases. With these three machines very economical results are achieved, even in relatively small batteries of retorts.

Coal Elevating and Conveying Plant.

IN connection with the retort-stoking machinery for gas works described in the preceding pages, a number of auxiliary machines are manufactured by Sir William Arrol and Company, Limited. These include coal breakers, coal elevators, coal conveyors, and wagon tips.

A wagon tip, coal elevator, and storage hopper are shown in the illustration opposite. The rear end of the wagon, it will be seen, has been raised by an hydraulic ram for the automatic discharge of the coal into a hopper underground, from whence it passes down inclined chutes to the boot of an elevator, which, in turn, raises it to the hopper on the high level. Where required, conveyors are also provided.

The elevator is driven by an electric motor of 5 horse-power, and can raise 20 tons of coal per hour.

Large installations of coal handling plant have been installed at various gas works and power stations, and deal with millions of tons of coal annually.

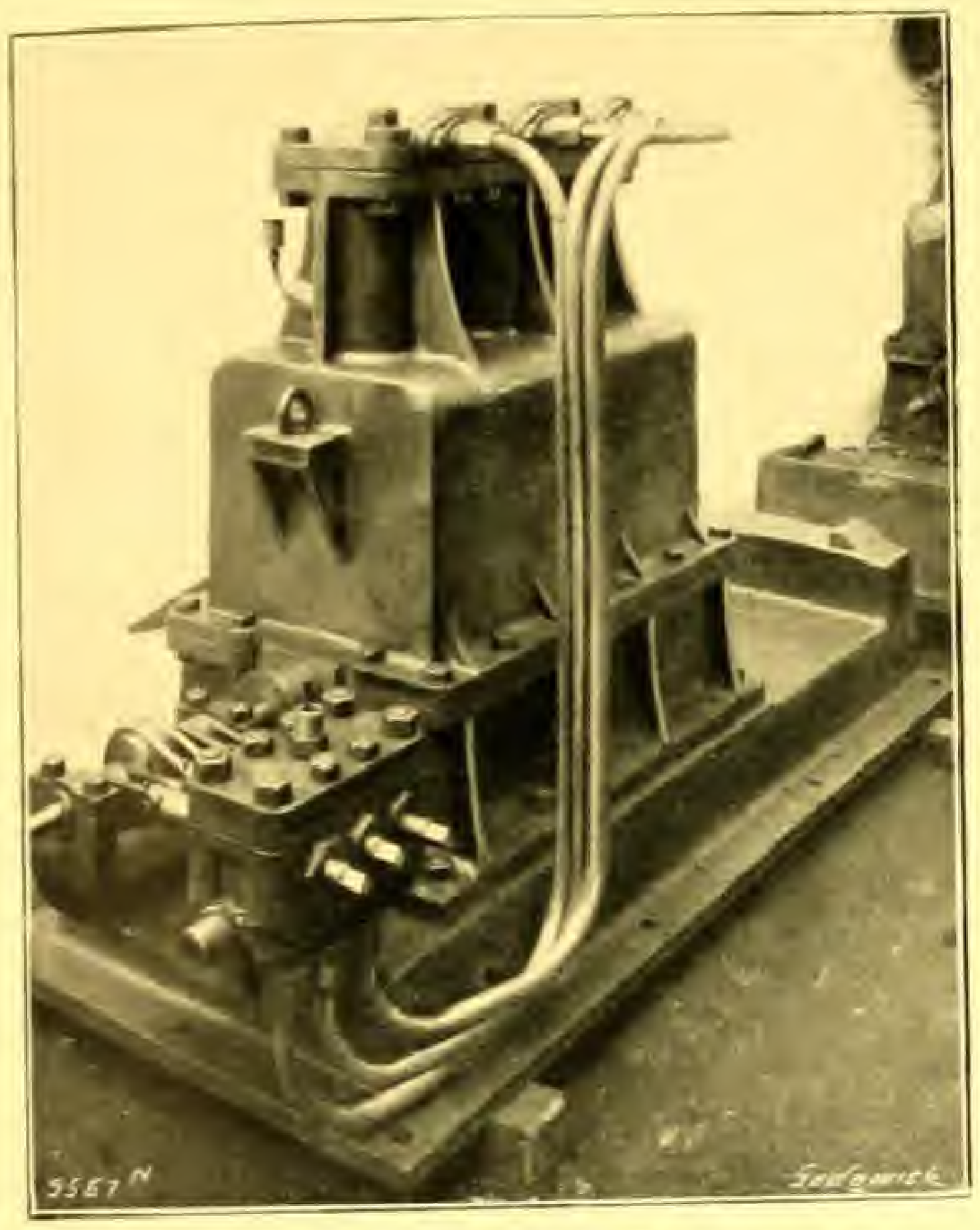
Coal Elevating Plant.

Hydraulic Motors.

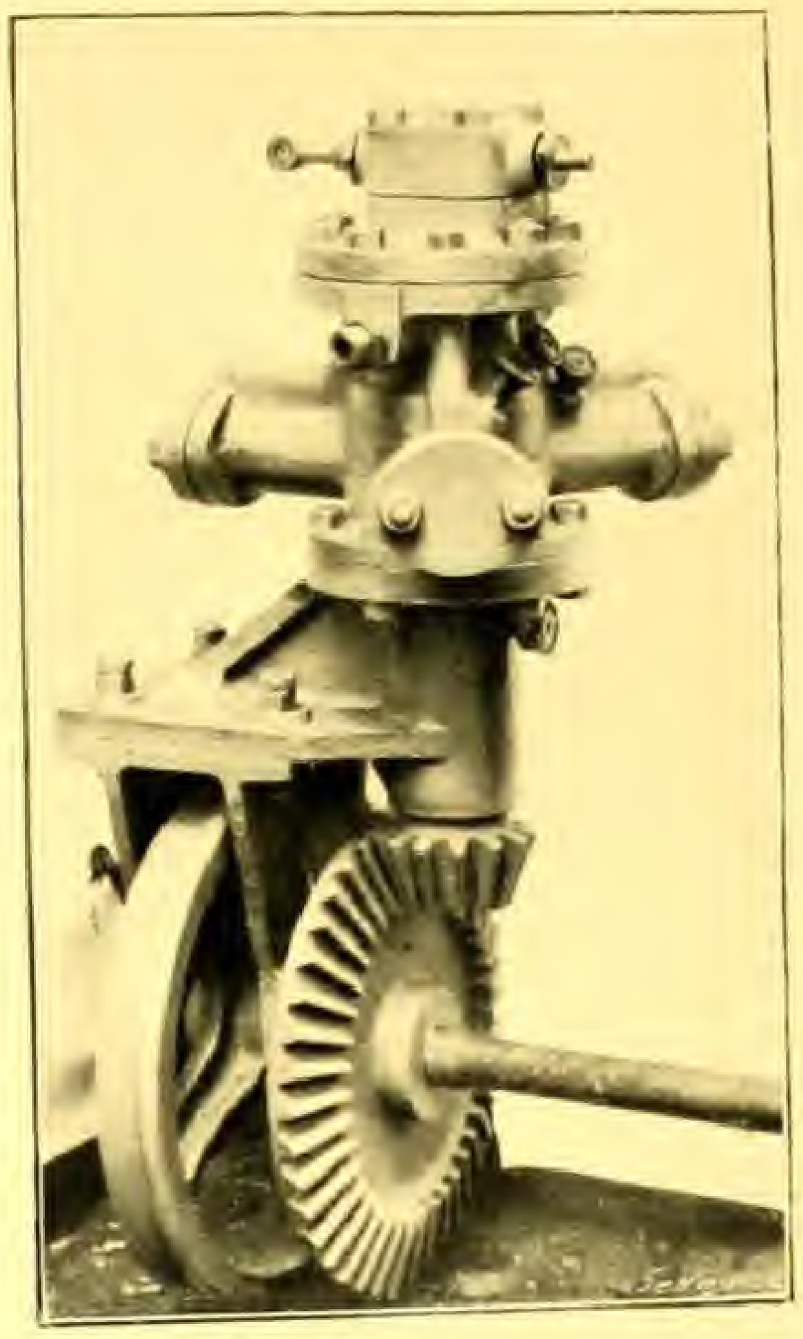
ON the opposite page we illustrate three types of hydraulic motors. The first is that used extensively for working conveyors and for slow speed driving generally. The cylinders, three in number, are formed along with the framing in one casting. The crank-shaft bearings are incorporated with the bottom and pedestal castings. The cylinders are lined with gun-metal, and the pistons are of the same material.

These motors are made in two sizes; (1) of 6 brake horse-power, in which case the cylinders are $4\frac{1}{4}$ in. in diameter and of 6-in. stroke; and (2) of 20 brake horse-power, having three cylinders, 6 in. in diameter and of 8-in. stroke. The motors work up to 60 revolutions per minute.

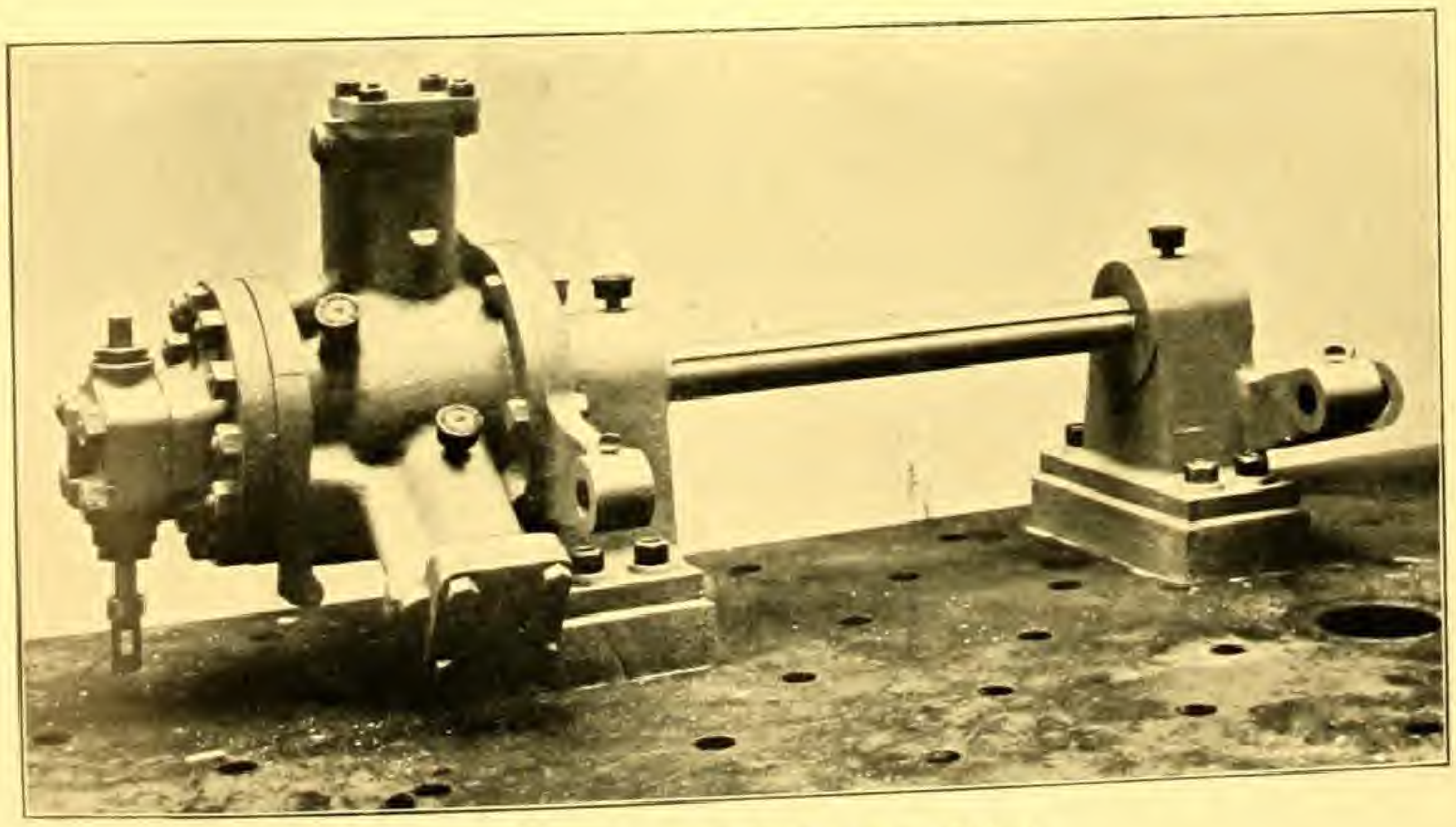
The second illustration shows a traversing motor with three cylinders set in one plane, and with all the pistons operating a single crank. These cylinders are $2\frac{1}{2}$ in. in diameter by 3-in. stroke. The machine has been brought to high efficiency, as the result of experience, especially in the traversing of gas-retort machinery. Similar machines have been used on the traversing carriages of riveting machines in shipbuilding yards where hydraulic power is available. The special feature is a patent reversing valve, by which the machine is reversed by a slight motion of an auxiliary sliding valve. Actuated by hand, it admits or exhausts water to or from the ports in the main circulating rotating



Motor for Working Conveyors.



Traversing Motor.



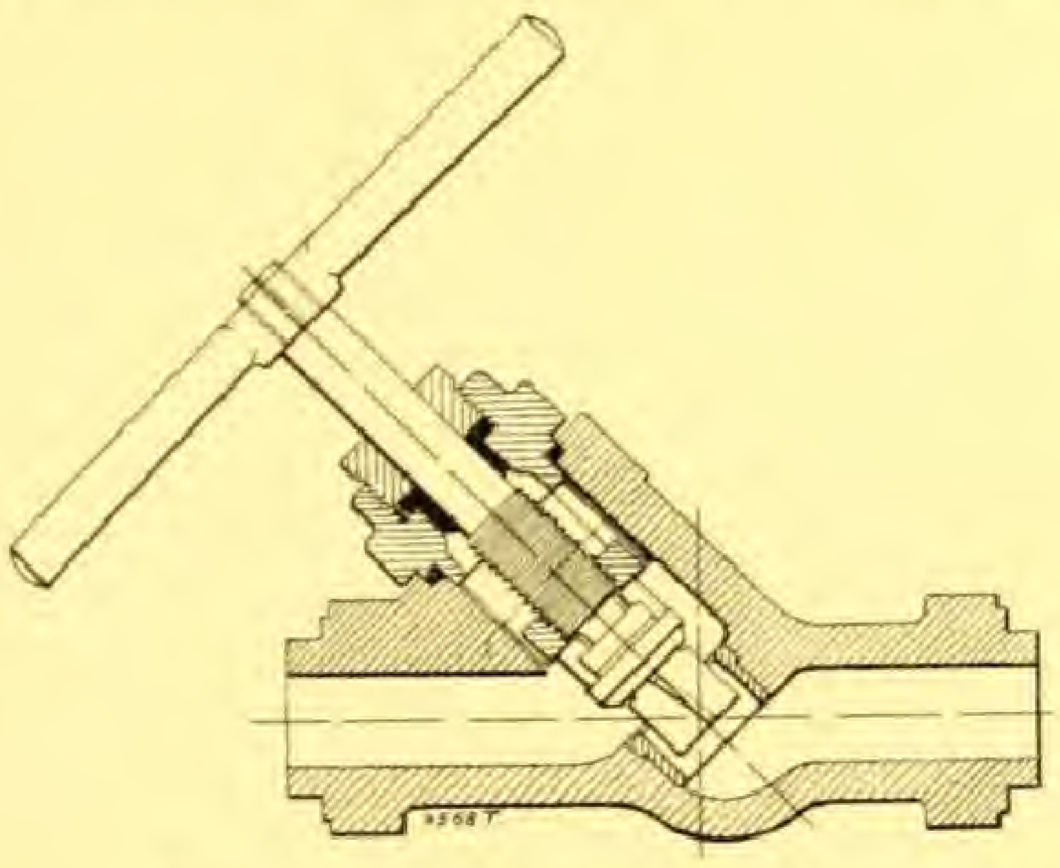
Capstan Type of Hydraulic Motor.

valve—a simple method of starting and reversing the machine.

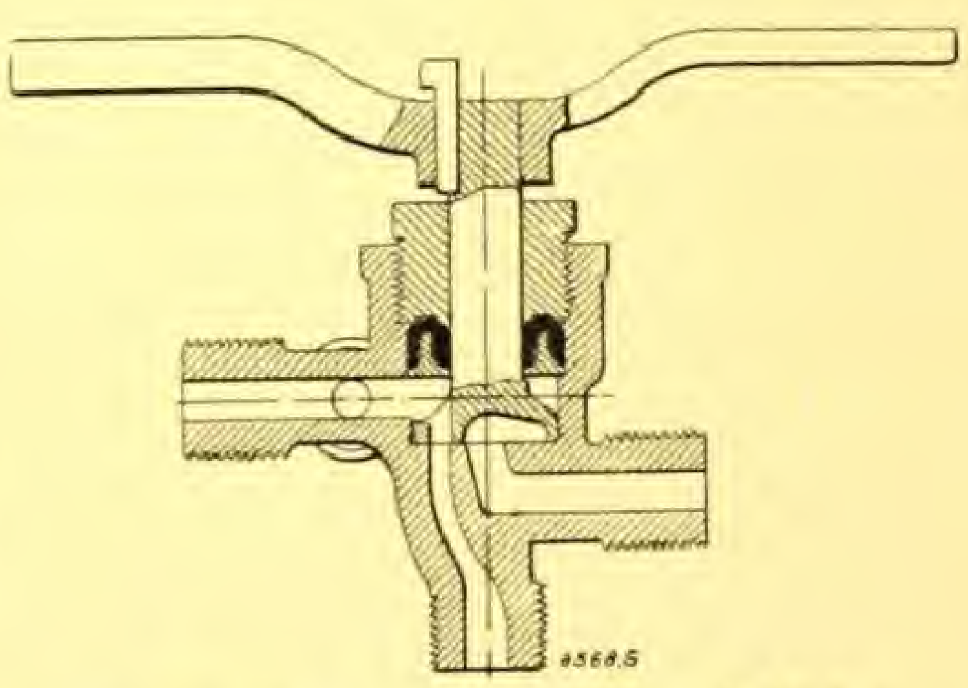
The third engraving illustrates a three-cylinder motor also of the capstan type, the connecting-rods working upon one crank-pin. In the illustration given on the preceding page, the motor is mounted on the axle of the working beam of a machine for cleaning ascension pipes; but it is made of many sizes, and is extensively adopted for many purposes. The cylinders in the machine illustrated are $3\frac{1}{2}$ in. in diameter by 4-in. stroke, and work with a pressure of 600 lb. per square inch at 60 revolutions per minute, which gives 4 brake horse-power.

Hydraulic Valves and Fittings.

THE success of hydraulic plant is influenced to a large extent by the efficiency and durability of the valves, fittings and details; and it is in this direction, as much as in any other, that Sir William Arrol and Company, Limited, are able to give clients the advantage of the



No. 1.—Standard Stop Valve.



No. 2.—Tapered Flat-Faced Valve.

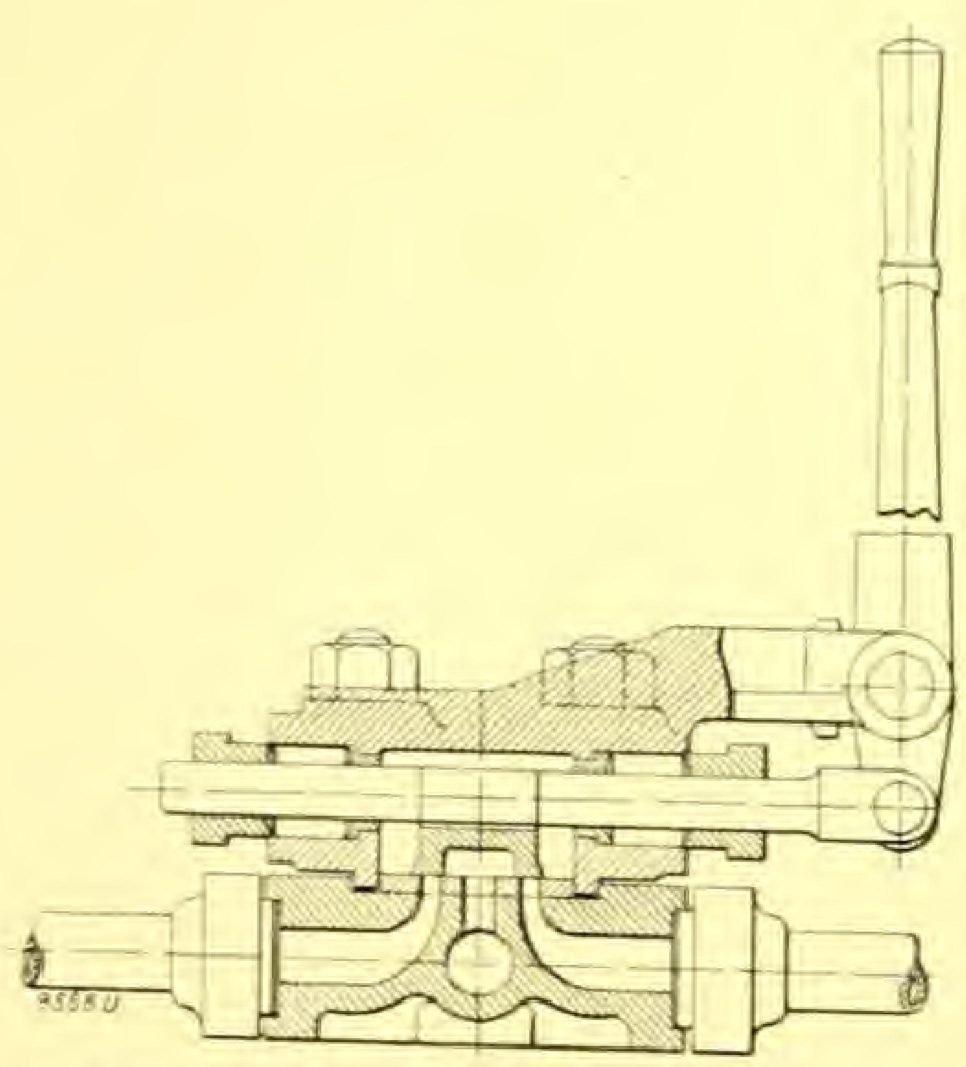
extensive experience gained in working their own hydraulic plant. A few typical fittings may therefore be described and illustrated, as they are made not only for the firm's own plant, but separately for other hydraulic installations.

The section marked No. 1 is a standard hydraulic stop-valve, generally used on all hydraulic plant made by the firm. The valve spindle is set at an angle of 45 deg. to the supply pipe, so that there is the minimum interference with the flow of water through the open valve.

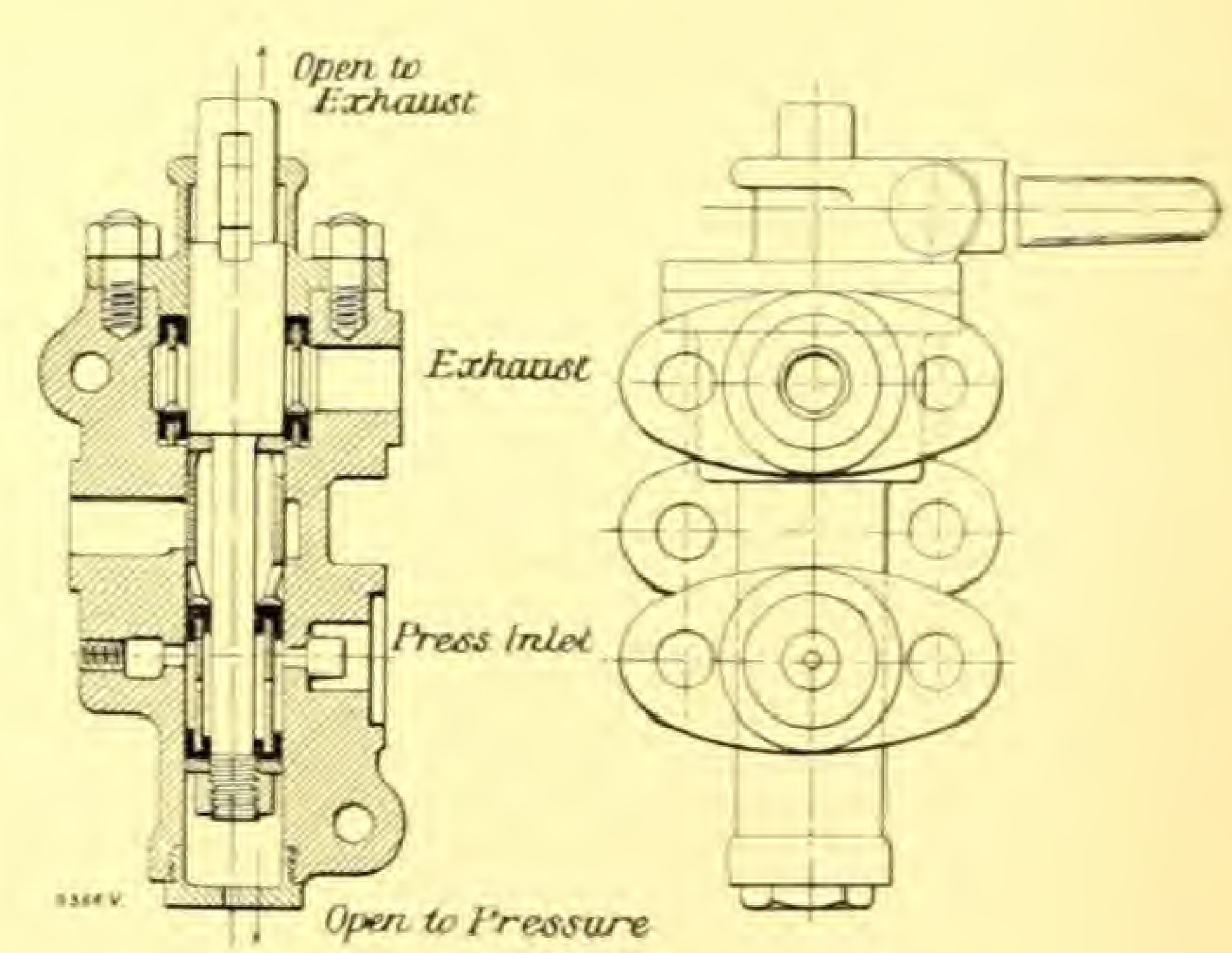
The Section No. 2 shows a flat-faced valve largely used on riveters.

A standard hydraulic slide-valve is illustrated by the Section No. 3. The lower portion is in gun-metal, with a hard seat sweated and pinned on; while the top portion is in cast iron. For low and medium pressures this type is satisfactory.

The elevation and section No. 4 illustrate a standard hydraulic piston-valve, which is made in large numbers by the Company.



No. 3.—Standard Slide Valve.

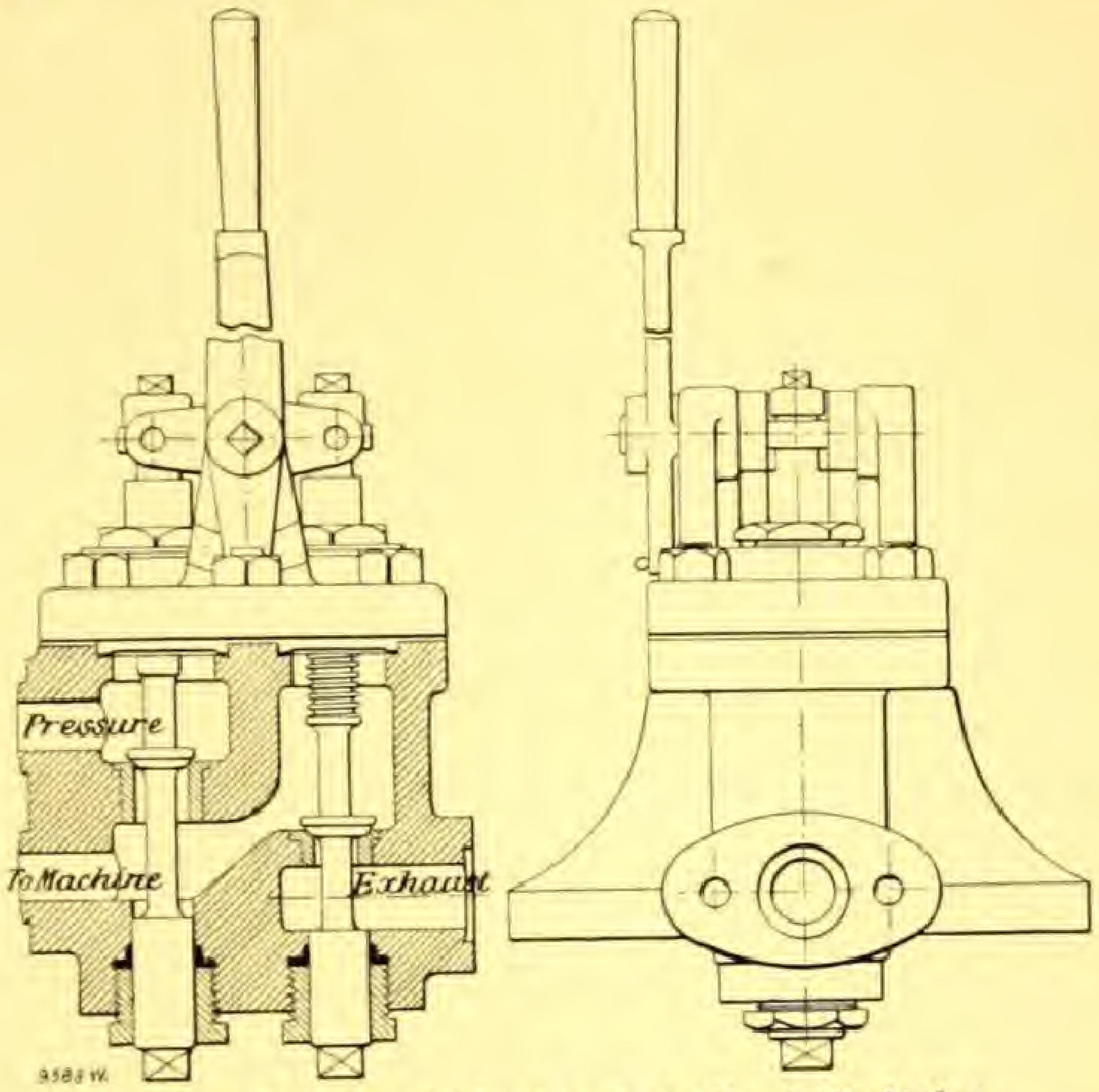


No. 4.—Standard Piston Valve.

It is balanced in all positions, and can therefore be actuated with great ease. At the same time there is little wear and no necessity for re-grinding. In the event of the working leathers requiring to be renewed, they can be replaced in a few minutes. The smaller valves up to 1 in. in diameter are made entirely of gun-metal; the larger valves have gun-metal liners in cast-iron shells. These valves are made in standard sizes, and the name-size represents the full-bore of the pipe to which they are fitted.

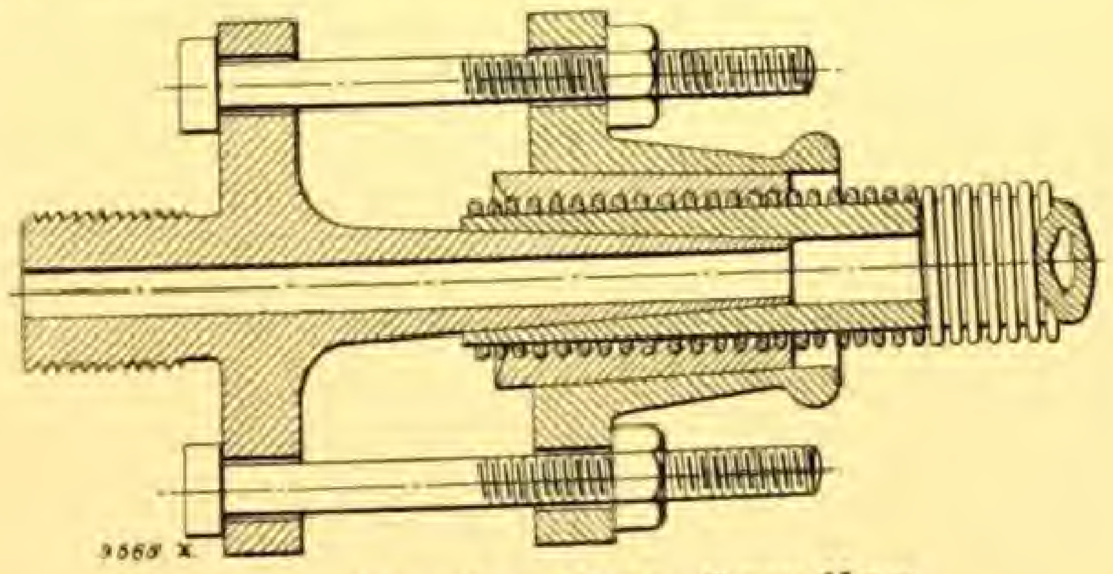
The elevation and Section No. 5 illustrate a standard

double-spindle lift-valve, which, being balanced, is rapid in its action. This valve is largely used on stamping presses to be described later.



No. 5.—Standard Double-Spindle Lift Valve.

The section No. 6 illustrates a patent hydraulic hose coupling for plain or wired hose, suitable for pressures up to 1500 lbs. per square inch. This coupling is made in standard sizes, and is extensively used.



No. 6.—Hose Coupling.

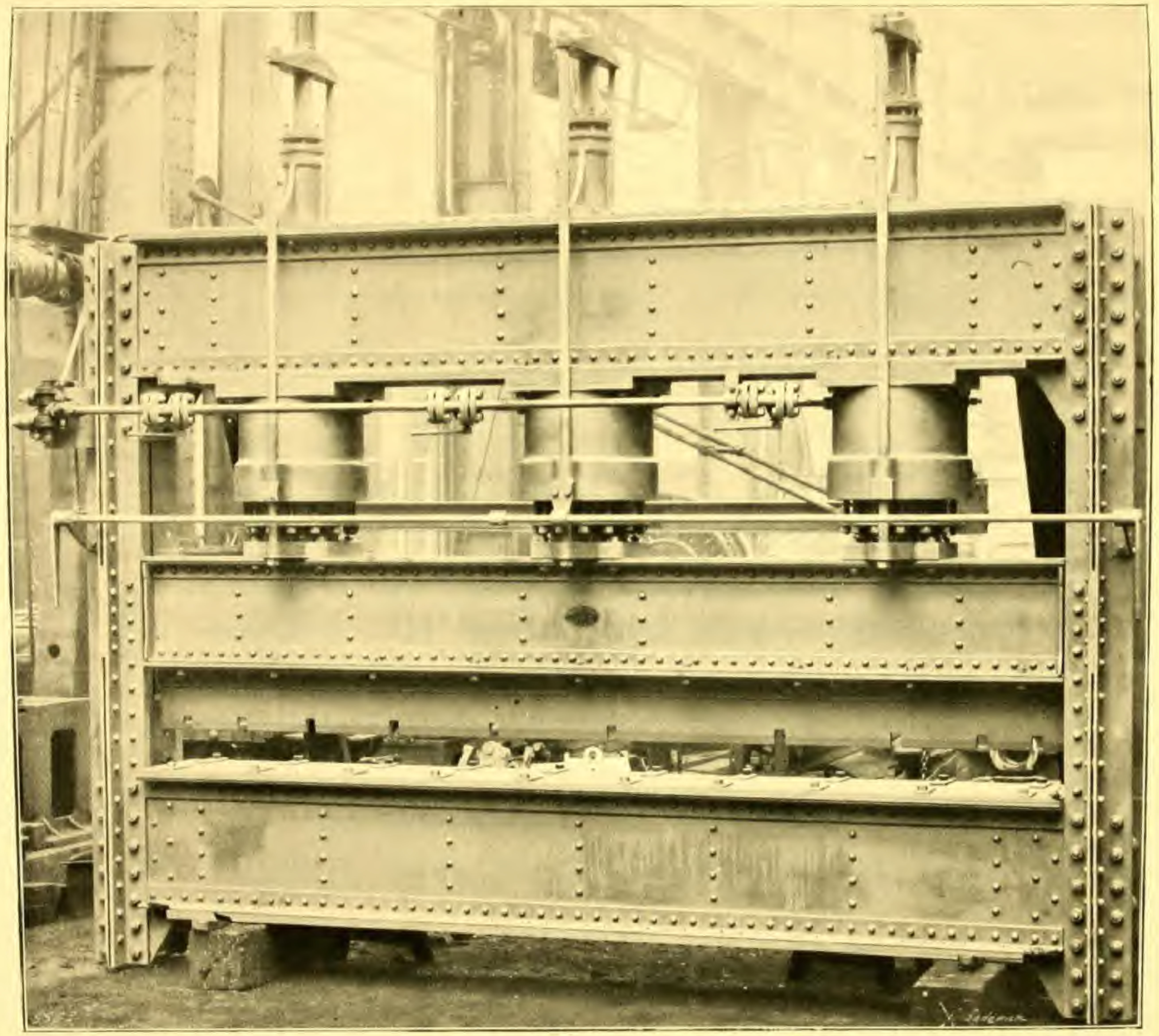
Sir William Arrol and Company, Limited, make a great variety of standard hydraulic valves and fittings; those illustrated are only types.

Hydraulic Stamping Press.

FOR stamping plates to any shape to increase resistance to stress, to make stiffening flanges, as in the case of water-tight bulkheads in ships, to form gutters, or to turn out corresponding sections, Sir William Arrol and Company, Limited, manufacture a special hydraulic tool, which is illustrated on the opposite page. This tool is made in various sizes—the one illustrated takes plates up to 15 ft. in width, and of any length. There are three hydraulic cylinders suspended from the cross girder forming the top member of the built-up structure, and each in this case is of a diameter of 15 in.

To the ram-heads there is secured a girder, which works in grooves formed by angles in the vertical members of the frame. This girder compensates for any variation in pressure in the cylinders. On the bottom of this girder there is bolted the dies for stamping the plates.

While all three cylinders may be operated simultaneously, a gun-metal cock has been fitted to each, so that any of the cylinders may be disconnected. In this way the total power to be exerted through the dies may be varied to suit the work in progress. Thus, with one 15-in. cylinder in action the pressure is 73 tons, with two cylinders 146 tons, and with all three about 220 tons. The working valve is at one end of the tool, with an actuating lever at each end for the convenience of the operator.



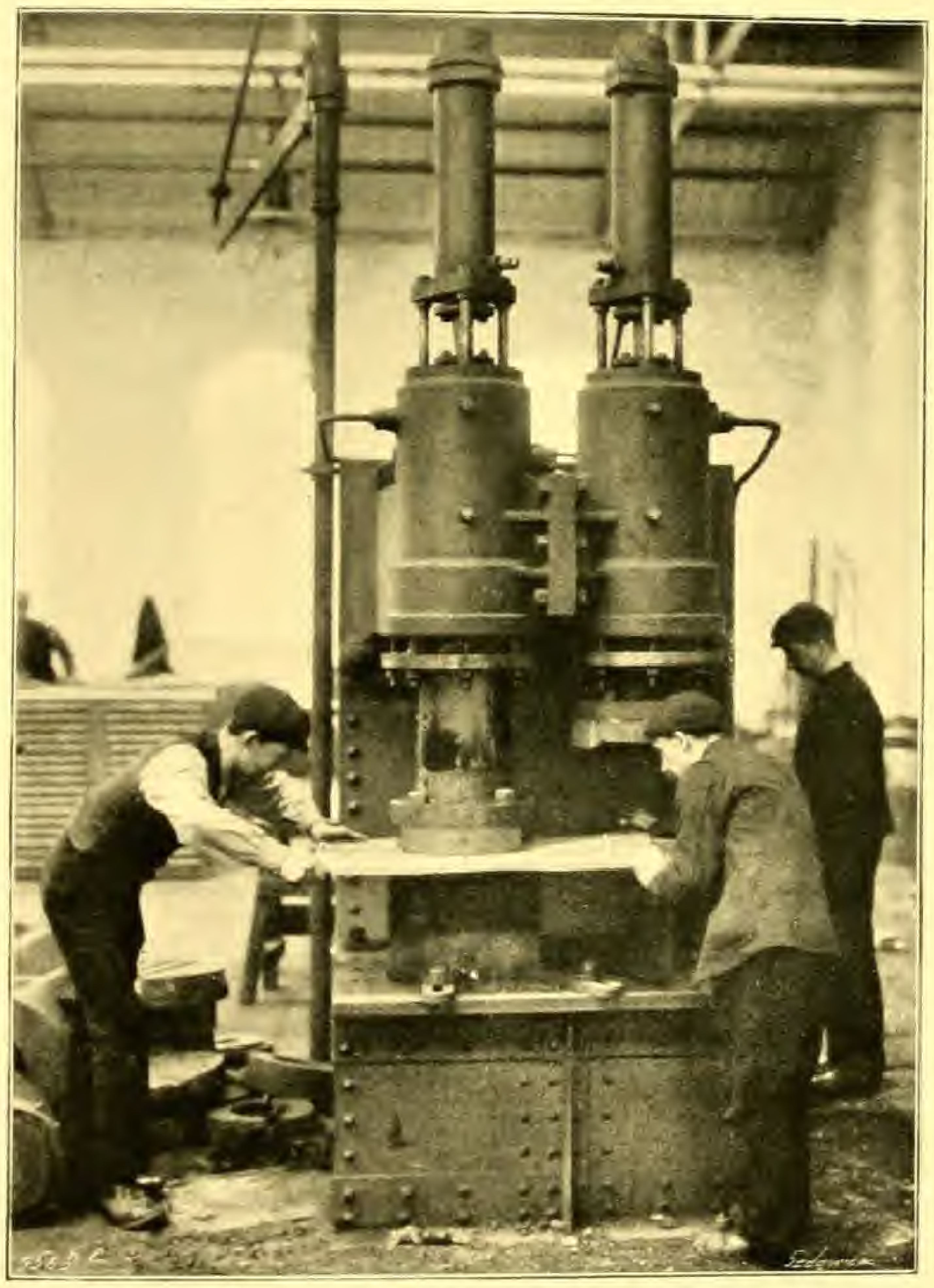
Hydraulic Stamping Press.

Light Power Stamping Press.

THE illustration opposite shows a light power stamping press, suitable for sheet metal and other light work.

There are two cylinders bolted to a vertical planed face on the built steel frame, and each cylinder is capable of exerting 30 tons pressure on the dies.

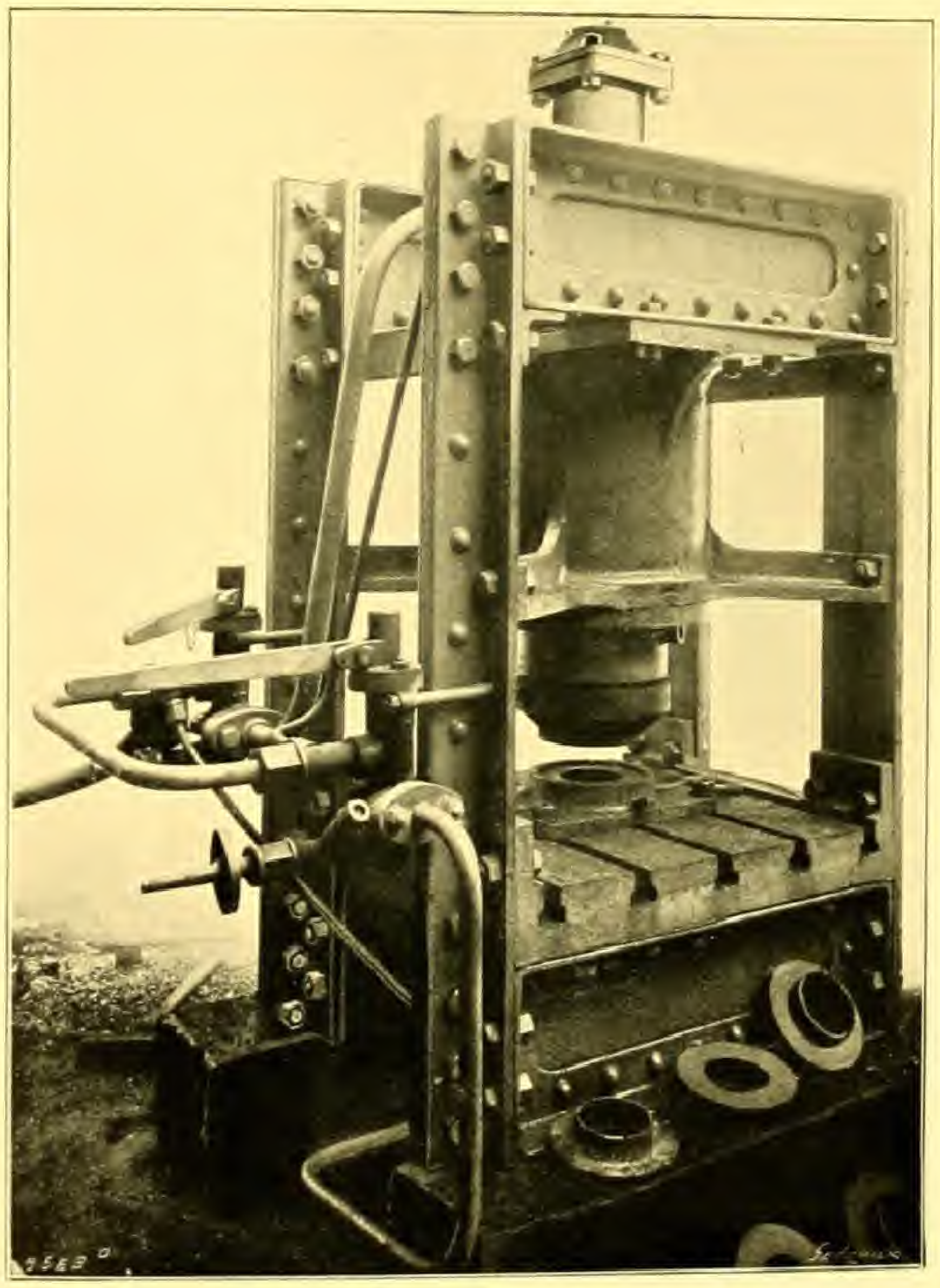
One ram can be used for holding the plate while the other does the pressing. The stroke of the ram is 15 in. There is a gap of 2 ft. to the centre of the cylinder, so that work of a bulky character can be undertaken.



Light-Power Stamping Press.

Hydraulic Flanging Press.

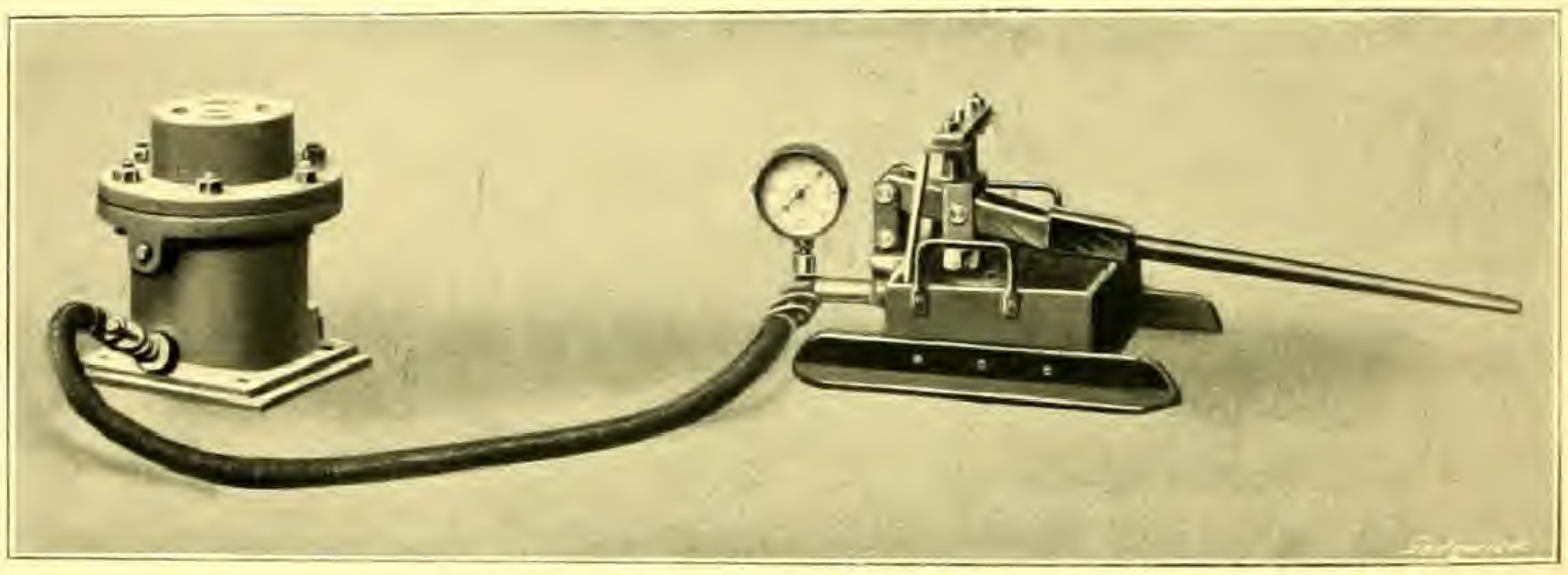
THE machine illustrated on the opposite page is intended for forming, by the use of dies, the rings or flanges of steel pipes cut out of a circular flat plate. The photograph reproduced shows that the cylinder, which is a steel casting with gun-metal linings, is secured by brackets to four columns carrying the entablature. The diameter of the cylinder is $15\frac{1}{4}$ in., and the stroke $3\frac{3}{4}$ in. In this cylinder there work two telescopic rams. The outer ram clamps the plate, while the inner, working within the main ram, and having a stroke of $11\frac{1}{2}$ in., turns down the metal to form the ring or flange. As a rule, two heats are required to carry out this work. The machine shown forms flanges for pipes up to 14 in. inside diameter.



Hydraulic Pipe Flanging Press.

Portable Hydraulic Jack.

THERE is illustrated on this page an hydraulic jack, with separate pan pump. This is extensively used for lifting heavy loads. Although the total weight of the whole appliance is only $3\frac{1}{2}$ cwt., it can move 50 tons through 9 in. As the jack is separate from the pump, it can be used in confined spaces, where the ordinary type of hydraulic jack could not be applied.



Portable Hydraulic Jack.

This portable plant is used for many varied purposes —for raising railway carriages and locomotives after they have been derailed, for straightening collapsed boiler flues, or for lifting heavy castings or girders into position, etc. The pump, which has a ram of $\frac{7}{8}$ in. in diameter, is worked by hand. The accessories to this equipment include a short length of flexible hose-piping, and two sets of gun-metal couplings. If high pressures are used, the connection between the pump and the jack is made by a solid-drawn copper pipe.

Hydraulic Press for Forming Knees and other Stiffening Units.

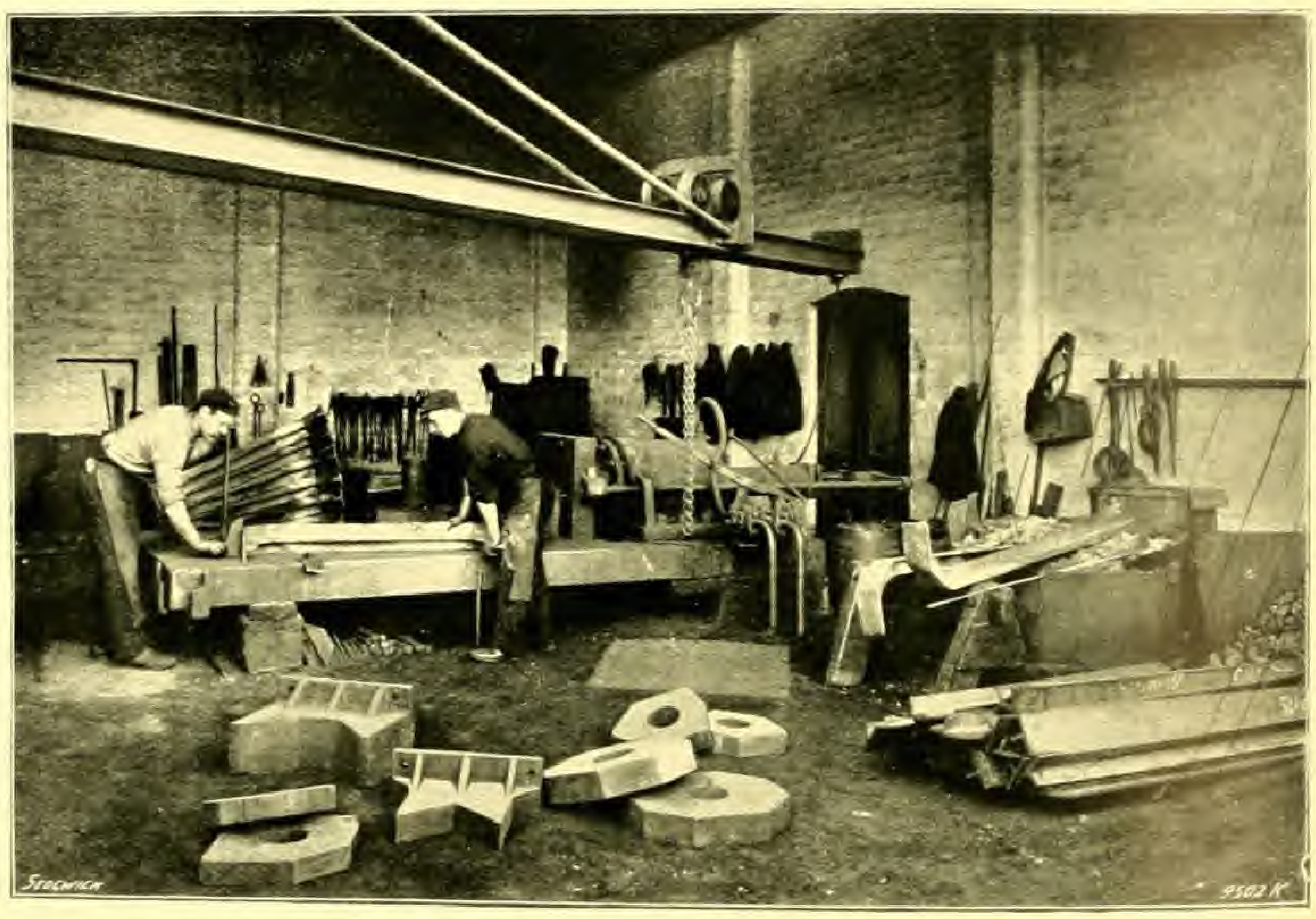
THE illustrations on this and the next page show an hydraulic stamping press, originally designed for

Hydraulic Machine for Forming Knee-Bars, etc.

forming knees, angles, and other stiffening pieces in the beams and webs of girders, but now very extensively applied in connection with corresponding details in ships, in locomotives, and many other manufactures.

This stamping press has a cylinder 14 in. in diameter, with a stroke of 18 in., and works at a pressure of about 1000 lb. per square inch. The cylinder is mounted on a horizontal massive table. On the ram head there are former blocks, while secured on the table in front are corresponding

dies. The bar is placed on the table between the blocks and the dies, and as the ram is forced forward by hydraulic pressure, the bar between it and the dies is squeezed into the exact shape required. The operation is expeditious and accurate. The whole of the metal within the bar is retained inside the knee, which becomes thicker and broader, and thus materially stronger.

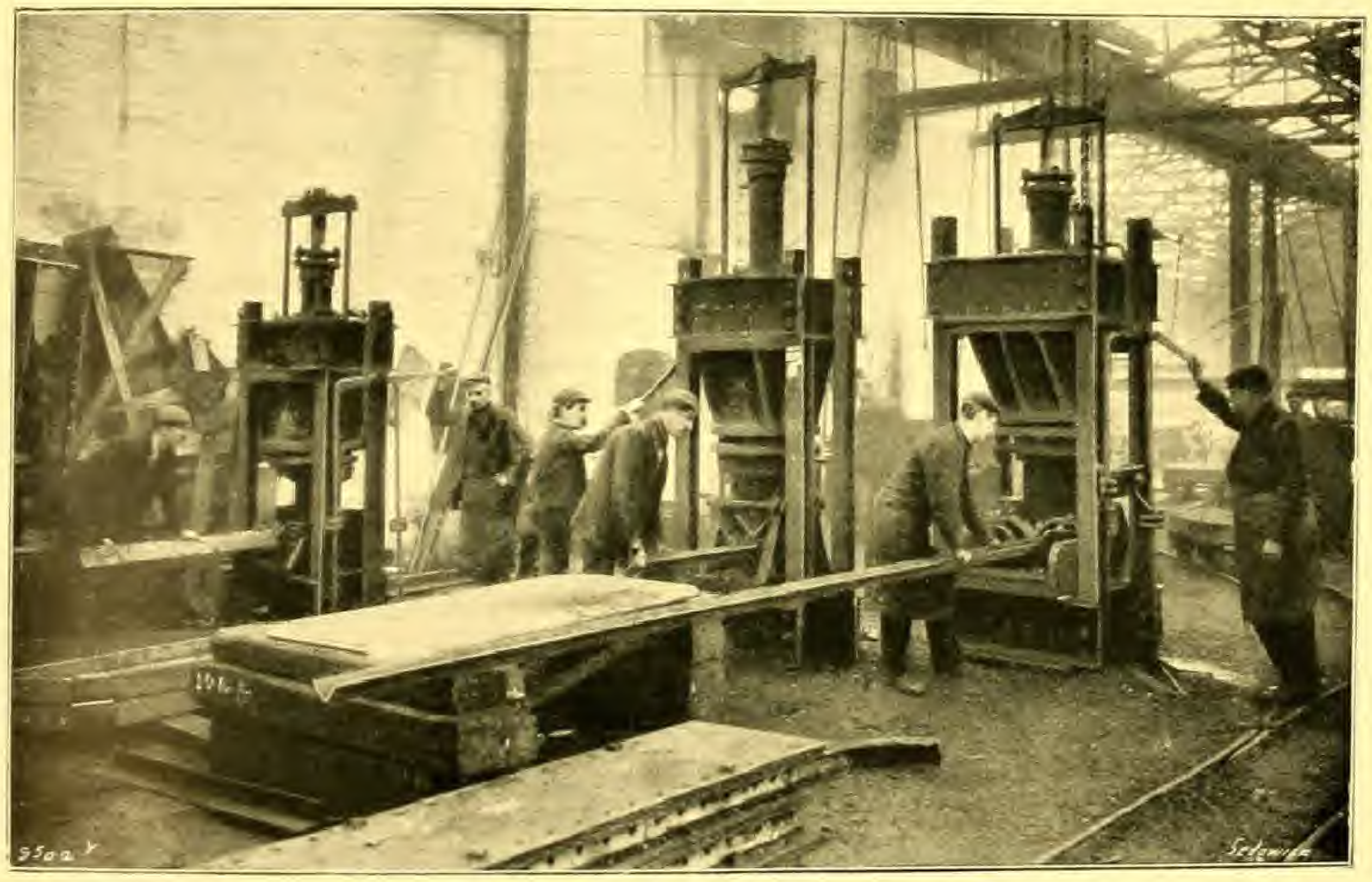


Machine and Dies for Forming Small Units of Girders.

As the moulds or dies can be made to suit any form, the machine may be utilised in the preparation of various details of structure, provided they are designed with a view to their production by the aid of dies. With each press there are supplied dies and blocks to suit the special work; and as the firm have succeeded in applying this tool to a great variety of work, suggestions can be given for its use in many classes of manufacture.

Hydraulic Angle-Cutting Machine.

THE illustration on this page shows a simple and effective hydraulic machine for cutting angles for ship and bridge



Cutting Bars and Angles for Girders.

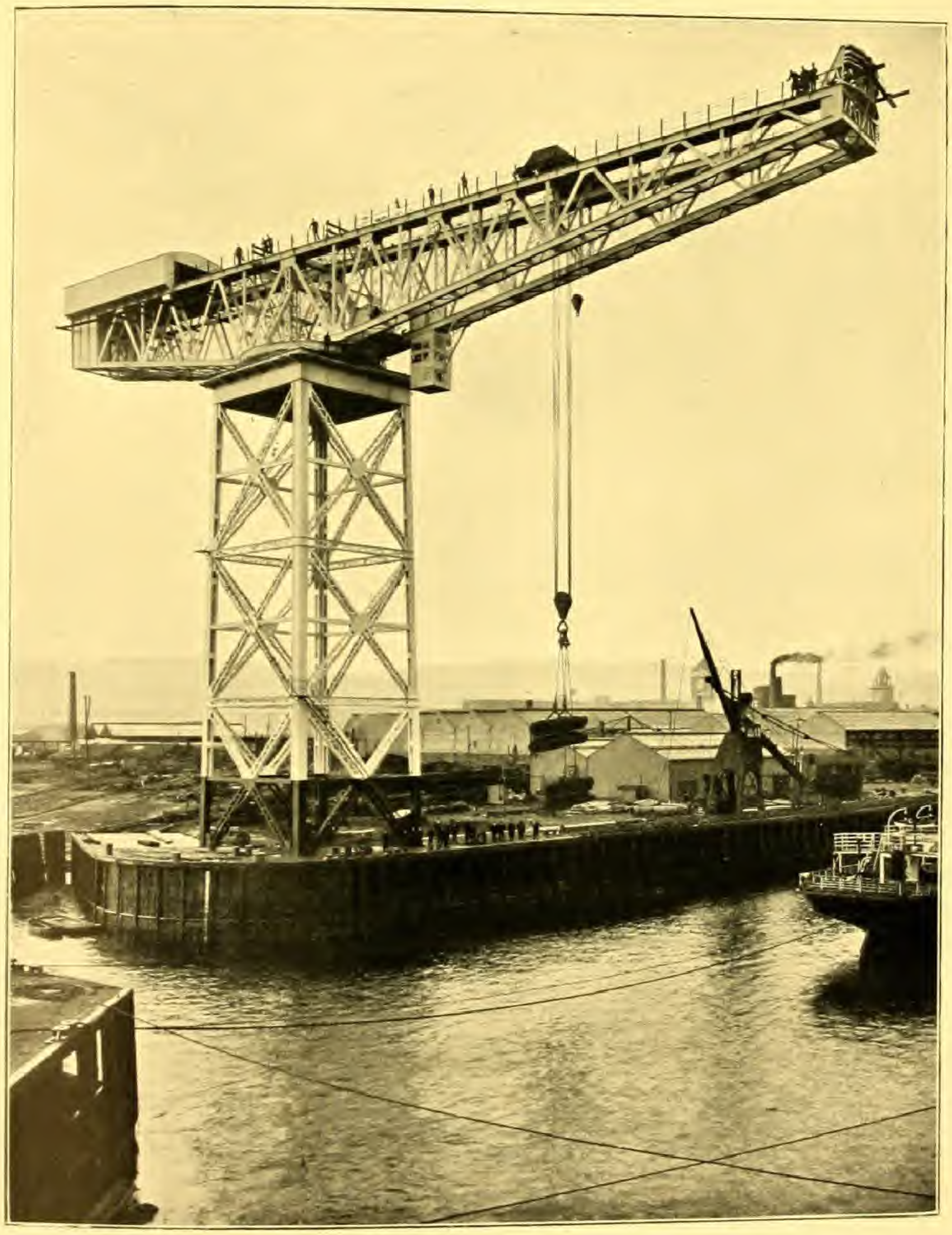
building. The shear is secured to a ram working in a cylinder, controlled by a simple form of valve. The angle rests on the anvil, which carries dies to suit the shape. By changing the dies, any form or dimension of bar may be cut, so that the tool has a wide range of application.

150-Ton Electric Hammer-Head Crane at Clydebank Works.[1]

THE great development of marine engineering in recent times has necessitated new appliances of special design to facilitate the handling of heavy pieces of machinery, large cylindrical boilers, framing of reciprocating engines, rotors and casings for powerful marine turbines, often weighing as much as 150 tons; and within the past two or three years what is now termed the hammer-head crane has in many instances displaced sheer-legs or jib-cranes. The earlier types had their origin on the Continent, and several such cranes of large power have been erected in Germany and Great Britain. These have generally proved satisfactory in solving the problem of handling the large weights, although, perhaps, wanting in some respects in that solidity which is desiderated by British ideas in the interests of durability and low cost of maintenance.

Messrs. John Brown and Co., Ltd., of Clydebank, have recently had completed for them two cranes of 150-ton capacity — one on each side of their fitting-out basin—so that they may deal with the heaviest loads for two ships simultaneously without moving the ships to and from the crane berth, as is done in other works, at

[1] The description of this crane is reproduced from ENGINEERING, vol. lxxxiii., page 737, where fuller particulars and detailed drawings are published.

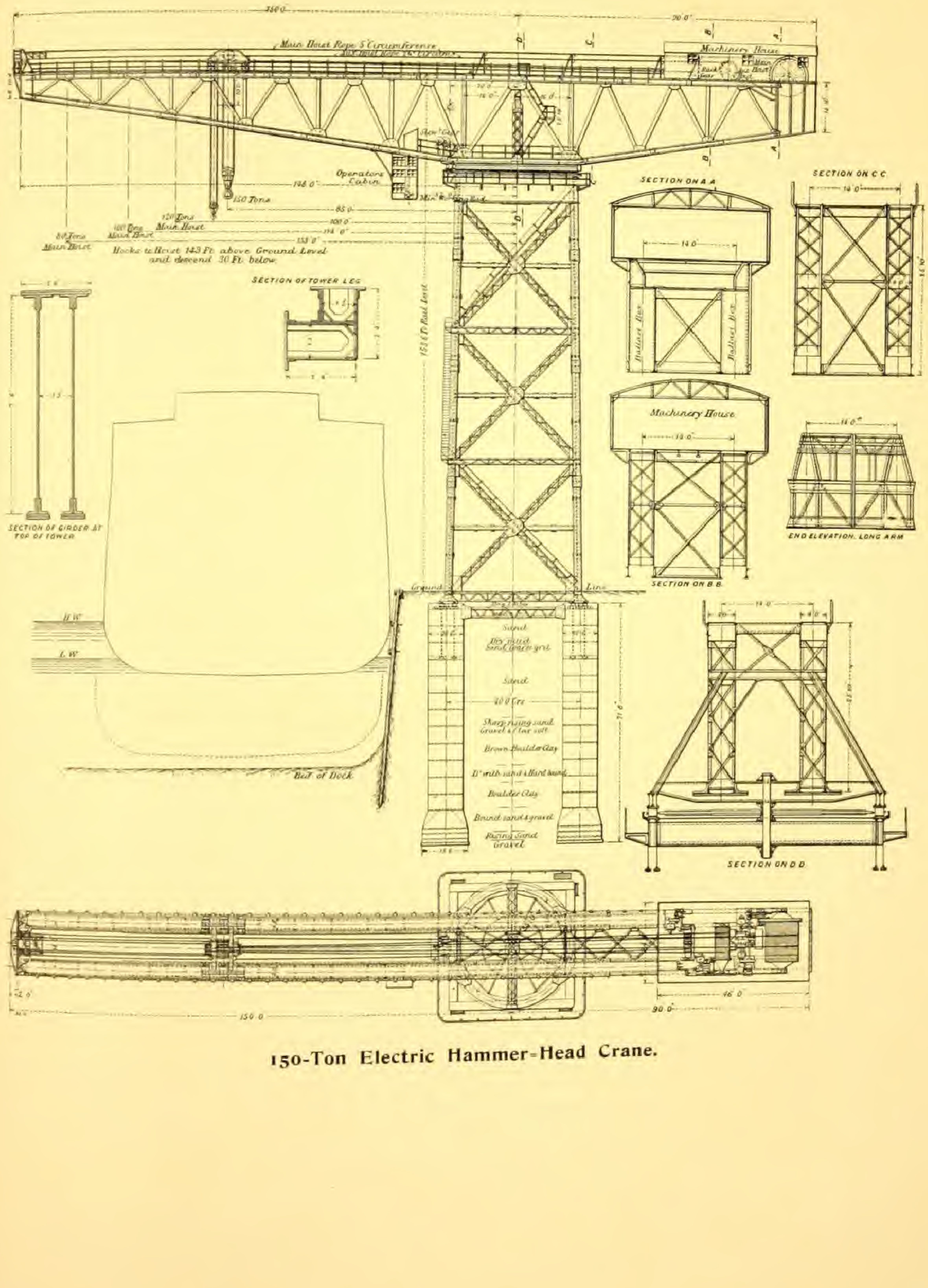


150-Ton Electric Hammer-Head Crane.

considerable expense in time and money. One of the new cranes is of the derrick type. The foundations are steel cylinders sunk to a great depth, and these, together with the structural portion of the crane, were built by, and from designs of, Sir William Arrol and Company, Limited. This crane is placed on the east side of the dock, and has been used in the fitting-out of the Cunard liner "Lusitania." The other crane is of the hammer-head or Titan type, and is placed on the west side of the dock. It consists of a square tower, 125 ft. in height, and carries a horizontal jib of a total length of 240 ft., the long arm being 150 ft. in length. The jib is supported upon a ring of live rollers, and is capable of making a complete revolution in both directions, with lifting and racking motions. This is the largest crane of its type yet completed.

The tower is formed of four legs, forming a square 40 ft. on each side of the base, tapering to 35 ft. at the top, with diagonal and horizontal bracing of box-girder section between them, and supporting at the top platform girders to carry the roller-paths and live ring under the jib. The foundations of the tower are formed of four cylinders, 75 ft. long and $10\frac{1}{2}$ ft. in diameter. The bottom is belled out to $13\frac{1}{2}$ ft. in diameter, to reduce the pressure on the ground to 6 tons per square foot, and to form a working chamber for the excavation of the material.

The platform supporting the live ring is constructed of four main box-plate girders, fitted between the legs of the tower, with short diagonal box-plate girders at each corner forming an octagonal frame under, and giving a continuous bearing to, the lower roller-path. The main platform girders are $7\frac{1}{2}$ ft. deep and $2\frac{1}{2}$ ft. in width, with a divided bottom flange, so as to permit of inspection and painting. Two transverse girders, crossing each other at right angles,



150-Ton Electric Hammer-Head Crane.

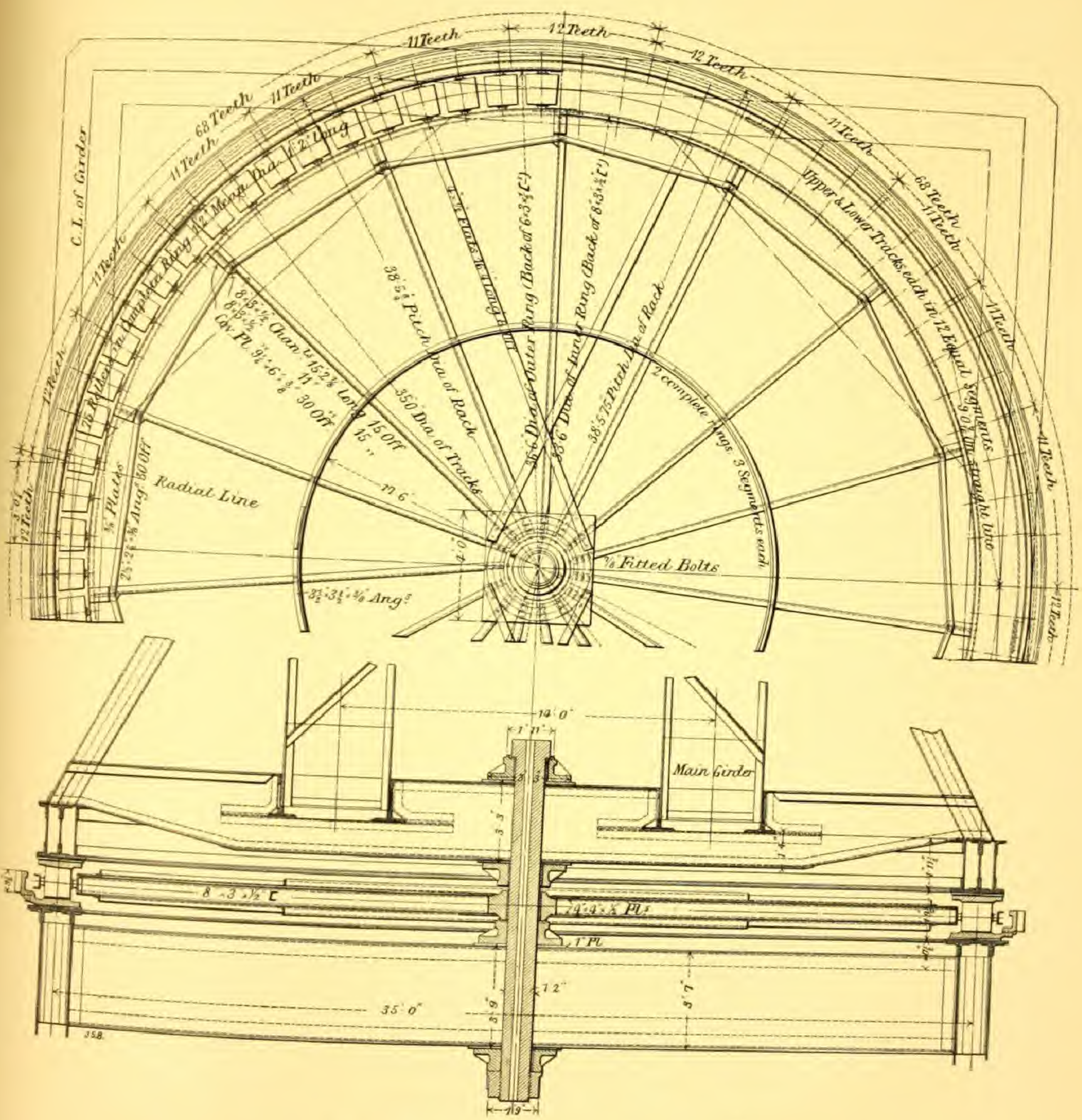
are placed between the centres of each parallel pair of main platform girders, to take the lower end of the centre pivot-pin. This pin is 14 in. in diameter and 13 ft. long, and passes through the cross-girders of the tower at their intersection, and through a box-girder secured to the main girders of the jib and the drum-girders.

Access to the crane is obtained by a stairway attached to the diagonal bracing on three sides of the tower and leading to a platform round the top of the tower, formed of $\frac{3}{8}$-in. chequered plates, with a suitable hand - railing. From this platform, stairs and gangways lead to the jib-rail level and operator's cabin.

The diameter of the track upon which the main girders rest is 35 ft., and there are seventy-five rollers in the live ring, each 14 in. mean diameter and 14 in. long. The rollers are forged steel, and are of conical form, and spaced about $16\frac{1}{2}$ in. centres apart. Drawings are reprodueed on pages 265 and 267.

The tracks are of cast steel, $2\frac{1}{2}$ in. thick, and are brought true by packing-plates and folding-wedges, which are inserted between the bottom track and the tower-girder, and between the top track and the annular bearing girder. The circular slewing rack is of $5\frac{1}{3}$ in. pitch, and is bolted to the palms cast on the lower track. A check is formed on the rack, and four forged hooks bolted to the annular bearing-girder, and revolving with the crane, engage with these checks, so that any possible upward movement of the girders, caused by the surging of the load, is prevented.

The jib is constructed of two open-lattice box girders, 14 ft. centres, the distance between the webs of each box-girder being 4 ft. The girders are 26 ft. deep over the tower, 7 ft. at the end of the long arm, and 15 ft. at



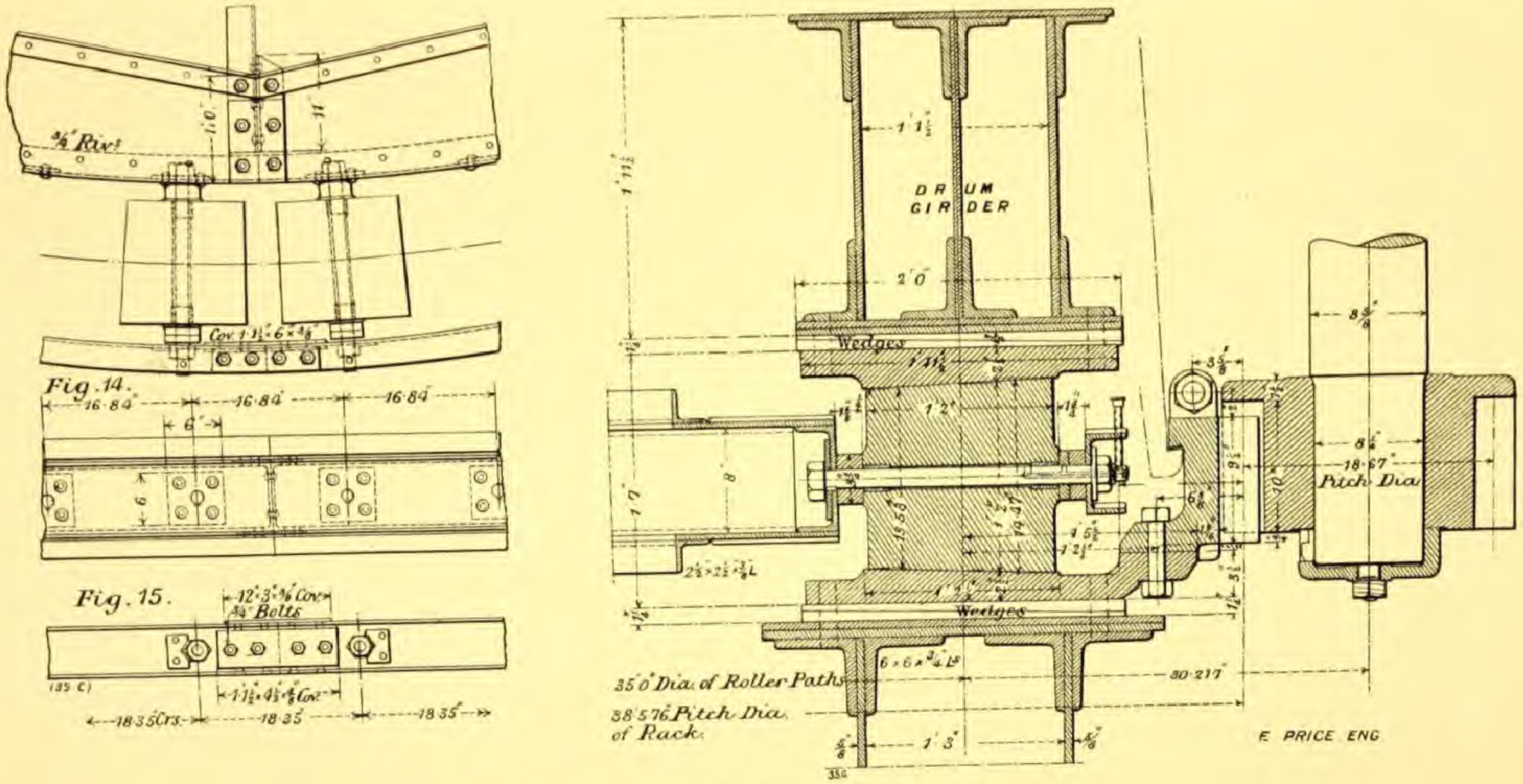
The Roller Track of 150-Ton Crane.

the ballast-box. The form of girder adopted was the open "Warren" type, with intermediate verticals from the intersection of the diagonals at their lower extremity to the top boom, thus breaking up the long panels of the top boom into smaller bays, to resist the severe bending stresses due to the concentrated wheel-loads of the jenny, which are additional to the direct tensile stresses as a cantilever. The box-girders are connected together at the end of the long arm by a stiff lattice frame, which carries the brackets supporting the horizontal girders to take the jib-head gear. From the front of the roller-path to the end of the short arm the two main girders are securely braced together, both horizontally and vertically. The ballast-tanks at the end of the short arm are filled with 86 tons of nickel slag concrete. Raking struts are placed between the centre vertical of the jib and the drum-girder, to transmit the lateral wind stresses to the tower and increase the lateral stability of the jib. The jib-girders are secured to the drum-girder by large gussets, of ample dimensions to resist the severe torsional stresses due to the sudden starting and stopping of the jib. Four lines of bridge rails are provided, one over each web of the box-girders, and they are riveted to the top flange.

The machinery is placed in a house formed of steel plating at the tail end, and assists in counterbalancing the crane. The speeds of the various motions are as follow:

Main lift	150	tons at 5 ft. 0 in. per minute
"	100	" 7 " 6 " "
Auxiliary lift	30	" 12 " 6 " "
"	$7\frac{1}{2}$	" 50 " 0 " "
Racking	150	" 40 " 0 " "
"	30	" 100 " 0 " "
Slewing	150	" one revolution in 10 minutes
"	30	" " 5 "

Messrs. Stothert and Pitt, Ltd., Bath, under a sub-

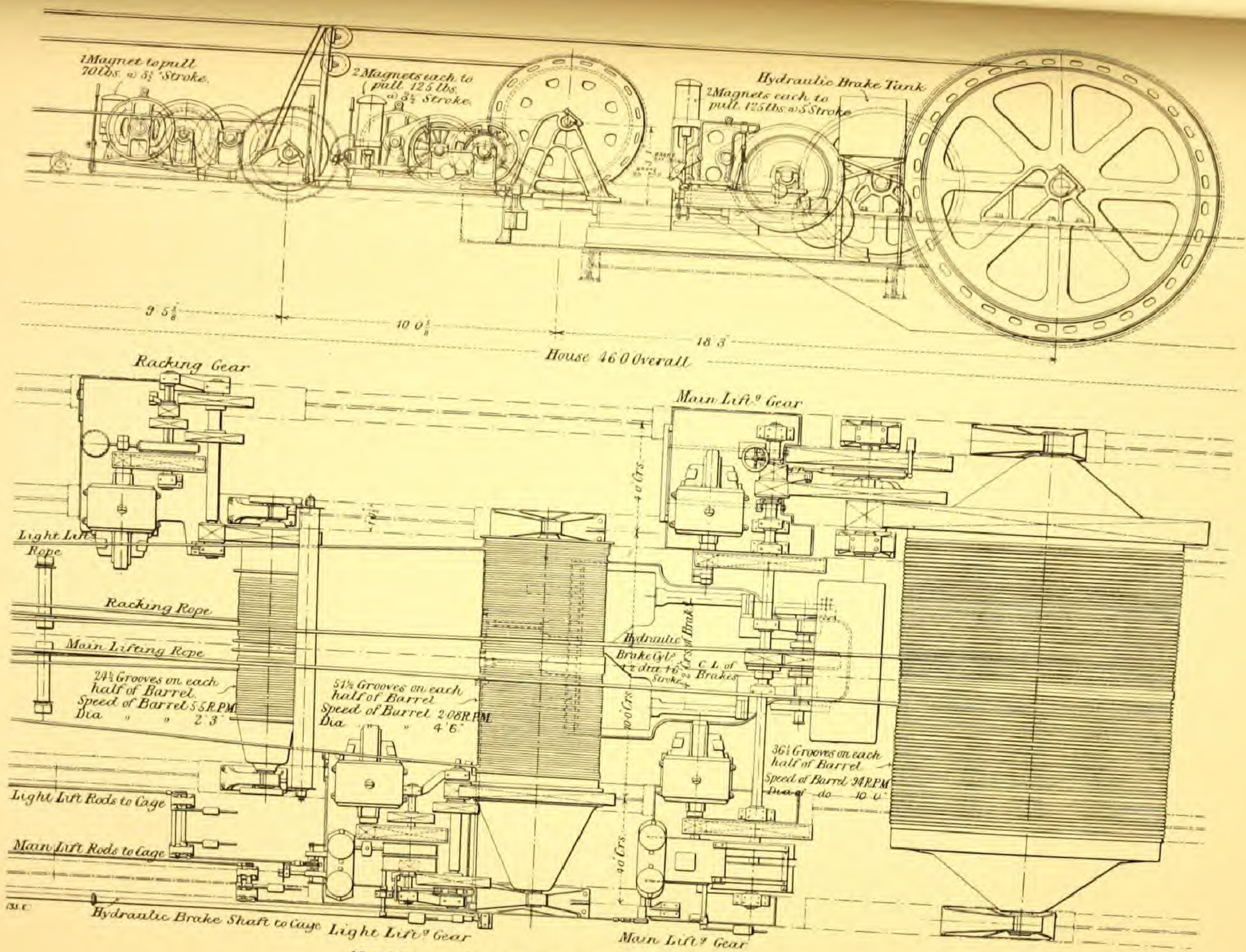


Details of Roller Track of 150-Ton Crane.

contract supplied the mechanical appliances connected with the lifting, racking, and slewing motions, together with the large travelling jenny, jib head-gear, ropes and snatch-blocks, with all working levers and equipment for the driver's cabin, and with all the collectors, wiring, and other electrical work on the crane.

The official test of the crane took place on April 24th, 1907, and was in every way satisfactory. The test load of 160 tons at a radius of 85 ft. was raised at a speed of 4.8 ft. per minute, and lowered by means of the hydraulic brakes, and successfully held by them and by the magnetic brakes independently. One revolution with this load at 85 ft. radius was made in $5\frac{1}{2}$ minutes. During the process of raising the load, observations were made of the vertical range of the vibrations by means of a wire suspended from the jib to a recording apparatus on the ground. The range varied from $\frac{3}{16}$ in. to $\frac{1}{4}$ in. The maximum deflection at the end under this load was $6\frac{7}{8}$ in., while the tail end of the jib rose $3\frac{1}{2}$ in. Under 80 tons load, at 133 ft. radius, the deflection was $7\frac{1}{4}$ in. With 30 tons load, on the auxiliary lift at the end of the jib, the crane made one complete revolution in 3 minutes 10 seconds, giving a velocity of nearly 300 ft. per minute at the end of the jib. The crane showed great rigidity and stiffness. The swing of the tower under the racking and lifting tests, and the twisting of the tower under the revolution of the loaded jib, were hardly perceptible.

The satisfactory completion of this crane is very creditable to Messrs. Sir William Arrol and Company, Ltd., and Messrs. Stothert and Pitt, Ltd. It demonstrates that British manufacturers are well to the fore in the design and construction of work of this special class and magnitude.



Machinery of 150-Ton Electric Hammer-Head Crane.

We append Sir William Arrol and Company's standard specification for the structural portion of heavy cranes:—

WIND PRESSURE.

(a) *Crane not Working.*—The wind pressure shall be taken at 50 lb. per square foot.

(b) *Crane Lifting Working Loads Causing Maximum Stresses.*—Wind pressure shall be taken at 5 lb. per square foot.

(c) *Crane Lifting Test Loads.*—No wind pressure assumed.

In all cases the wind pressure shall be assumed to act on a surface equal to $1\frac{1}{2}$ times the area of the surface seen in the elevation.

MOMENTUM, IMPACT, &c.

The stresses caused by the slewing and stopping of the jib, the lifting and racking of the various loads at their respective speeds, the effects of the brakes, &c., shall be provided for in proportioning the sectional areas required for all parts of the structure.

MAXIMUM PERMISSIBLE STRESSES UNDER DEAD-LOAD AND FULL WORKING LOADS.

Jib and Tower.—The maximum stress shall not exceed $6\frac{1}{2}$ tons per square inch on the net section in tension or compression, but in no case shall the member in compression be subjected to a greater stress than one-fifth the ultimate strength of the member considered as a column. The stress on the rivets shall not exceed 5 tons per square inch in shearing and 10 tons per square inch in bearing.

In members subject to stresses from wind pressure only, the stress shall not exceed $7\frac{1}{2}$ tons per square inch on the net section in tension or compression; but in no case shall the member in compression be subjected to a greater stress than one-fourth the ultimate strength of the member considered as a column. The stress on the rivets shall not exceed 6 tons per square inch in shearing and 12 tons per square inch in bearing.

ALTERNATING STRESSES.

Members subject to alternate tension and compression shall have sectional areas equal to the joint areas for the compressive and tensile stresses considered independently, except in the case of wind-bracing, where the additional sectional area may be one-half the preceding.

BENDING STRESSES.

Where a bending stress occurs on a member subjected to a direct tensile or compressive stress, the sectional area shall be proportioned to the sum of the stresses.

ROLLERS.

The pressure in pounds per lineal inch of live rollers shall not exceed that given by the formula $300d$, where d = diameter of roller in inches.

JOINTS IN MEMBERS.

Tension.—Joints in tension members shall be fully covered and riveted to resist the maximum tensile and shearing stresses transmitted through the joint.

Compression.—The butting ends of compression members shall bear through their whole faces, and be covered and riveted to a sufficient extent to transmit at least one-third of the thrust as a shearing-stress through the rivets. In plate-girder top booms the proportion shall be two-thirds.

SECTIONAL AREAS.

In estimating the sectional area of the flanges of the jib, the rails and floor plating shall not be included in the area; for net sections the diameter of the

rivet-holes shall be taken as $\frac{1}{8}$ in. larger than the nominal diameter of the rivet before driving. In plate girders one-eighth of the web plate shall be included in the flange area.

Foundations.

The pressure on the concrete shall not exceed 10 tons per square foot under the working loads and wind pressure.

Constructional Details.

All members shall be designed of such form as to be accessible for inspection and painting, and to allow a free circulation of air through the member.

To allow for corrosive influences in the vicinity of manufacturing districts and near the sea, no angle, plate, or bar shall be used of a less thickness than $\frac{3}{8}$ in.

To minimise vibrations, and to provide a substantial and rigid frame, all members of the crane shall be formed of sections capable of resisting tension or compression, and the diagonal bracing of the tower shall be designed as tension members only.

The diameter of the roller-path shall be such that, under all conditions of loading, the centre of gravity of the loaded jib shall not approach nearer the centre of the roller-path than one-tenth of the diameter.

All radial bars and roller frame of the live ring shall be formed of rigid members, with suitable tangential bars to maintain the relative motion of the parts of the frame.

To provide for emergency loading, and a construction of a substantial and durable character, the centre pin, with the girders carrying it, shall be designed to resist a vertical and horizontal force of at least two-thirds of the maximum load lifted by the crane.

The size of the tower in plan shall be such that no tension shall exist in the foundations.

Maximum Permissible Stresses under the Test-Loads.

The sectional areas for the different members of the jib and tower shall not be increased unless the stress per square inch exceeds the specified stresses by 25 per cent.

Materials.

Wrought Steel-Work.—The whole of the steel-work, unless otherwise distinctly specified, shall be of steel, made by the Siemens-Martin open-hearth acid process, having a tensional strength of from 27 to 32 tons per square inch, with an elongation of at least 20 per cent. in a length of 8 in.

Rivet steel shall have a tensional strength of 26 to 30 tons per square inch, and a minimum elongation of 25 per cent. in a length of 8 in.

Cast steel shall be practically free from blow-holes and other defects, and is to be annealed in all cases, and have a tensional strength of from 27 to 32 tons per square inch, with an elongation of 13 per cent. in a length of 8 in.

Workmanship.

The whole of the workmanship shall be of the highest class. Where stress is transmitted the sheared edges of plates and bars shall be planed, and all holes drilled. Hydraulic or power-riveting shall be used where practicable. The abutting faces of compression members shall be machined after the section is riveted, so as to ensure perfect contact on the abutting surfaces.

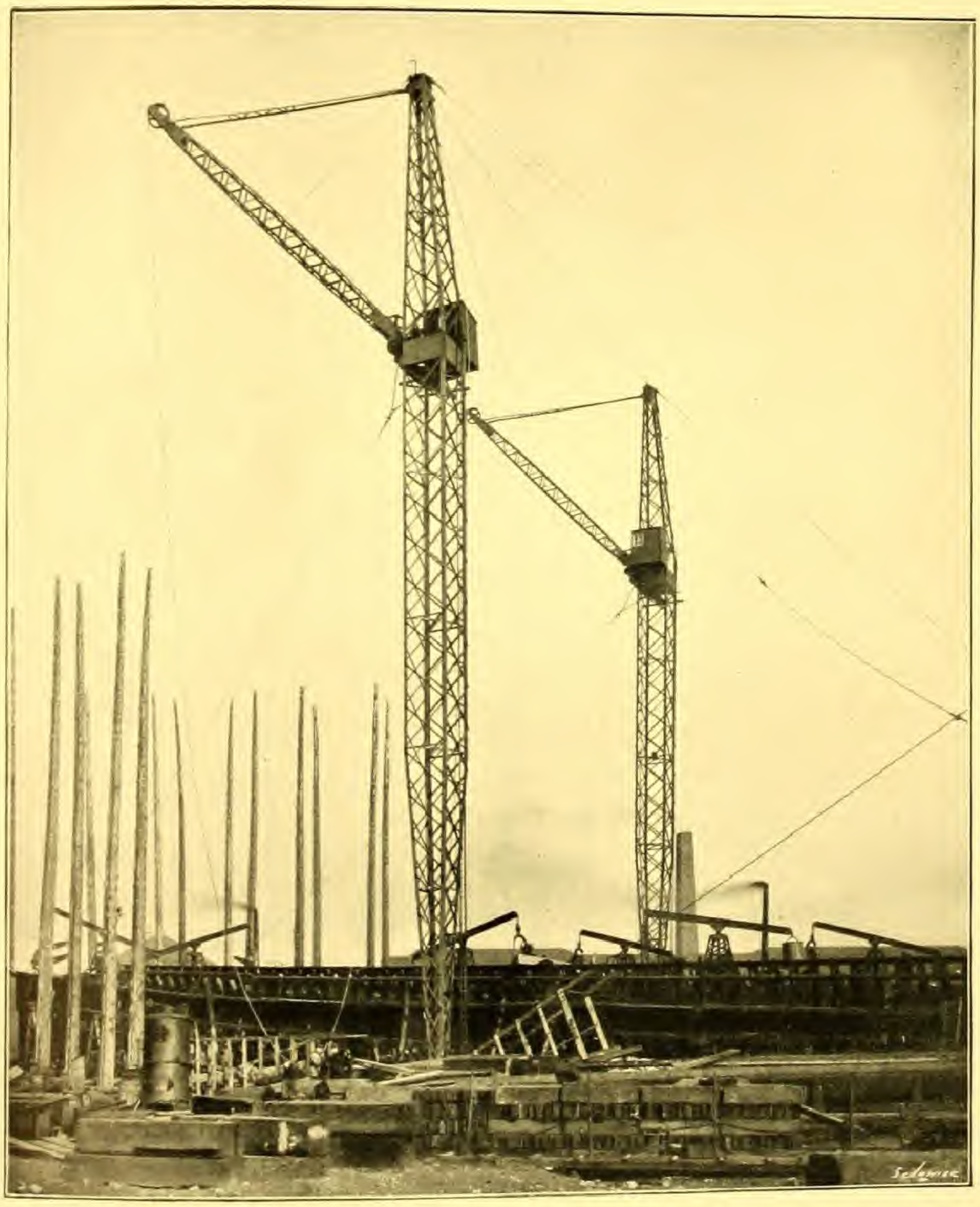
Electric Derricks for Shipbuilding Berths.

THERE are illustrated on opposite page electric derricks of great height, which were constructed for use in the building of the express Cunard liner "Lusitania" at the Clydebank Works of Messrs. John Brown and Co., Ltd. This ship, as is well known, is the largest that has ever been constructed; and, in view of its great height, it was necessary to arrange for exceptional derricks.

These were designed to lift a load of 5 tons to a height of 120 ft. from the ground level, with a working radius of 35 ft. from the centre of the mast, and with a jib slewing through 180 deg. The mast is constructed as an open lattice-work column of square section, with four corner angles, well braced together. It is 6 ft. square at the central portion, and tapers to 18 in. square at the ends. Four guys are attached to the top of the mast, and one underneath the jib. A platform for the electrical gear is arranged at a height of 95 ft. above ground level, and from this point there is carried the jib, which is rectangular in section.

The jib is set at an angle of 45 deg., and at the outer end there is fixed the usual pulley for the lifting rope. This rope passes over a deflecting pulley at the base of the jib, and thence through the centre pin to the electric winch located on the ground level.

The electric motor, of 30 brake horse-power on the winch, is arranged for two speeds of lifting—90 ft. per



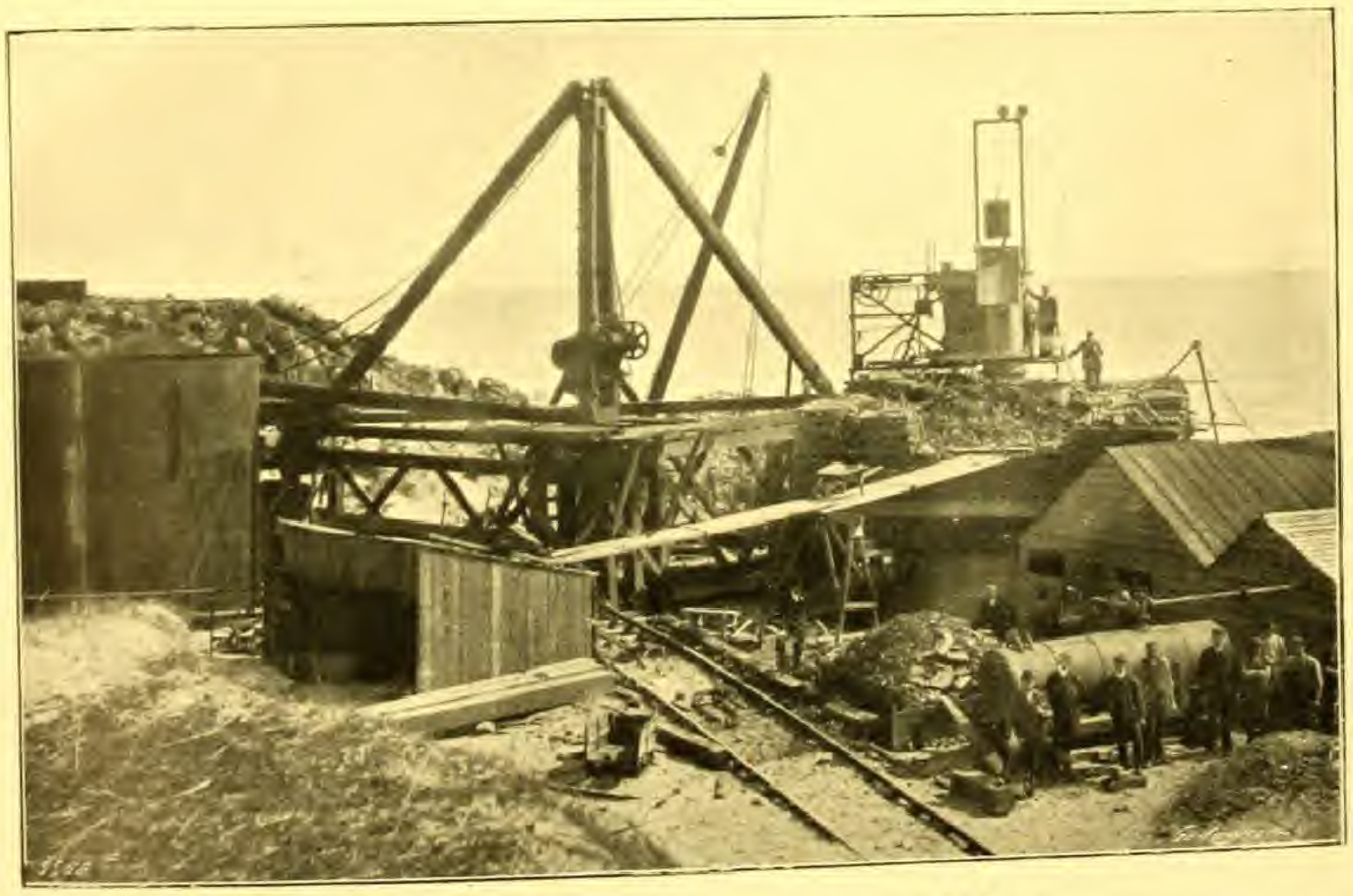
Electric Derricks for Shipbuilding Berths.

minute and 210 ft. per minute, the former being for the full load of five tons. The slewing motor is placed on the working platform, 95 ft. from the ground level.

The view shows two such derricks, built one on each side of the Cunard liner, the double bottom of which is in process of construction. Six riveting machines are shown at work on the double bottom, all of Arrol's make.

Compressed=Air Plant for Sinking Piers and Shafts.

FEW firms, if any, have done so much work in the sinking of bridge and harbour piers, etc., under compressed air, as Sir William Arrol and Company,



Sinking a Pit Shaft by Compressed Air.

Limited. In the work of sinking the 70-ft. diameter piers of the Forth Bridge, of the 10,500-ton piers of the Wear Bridge, of the 117-ft. deep piers of the Barrow Bridge in Ireland, as of many equally important caissons, great experience has been won; and this is applied in

the plant manufactured at the Glasgow Works for such operations.

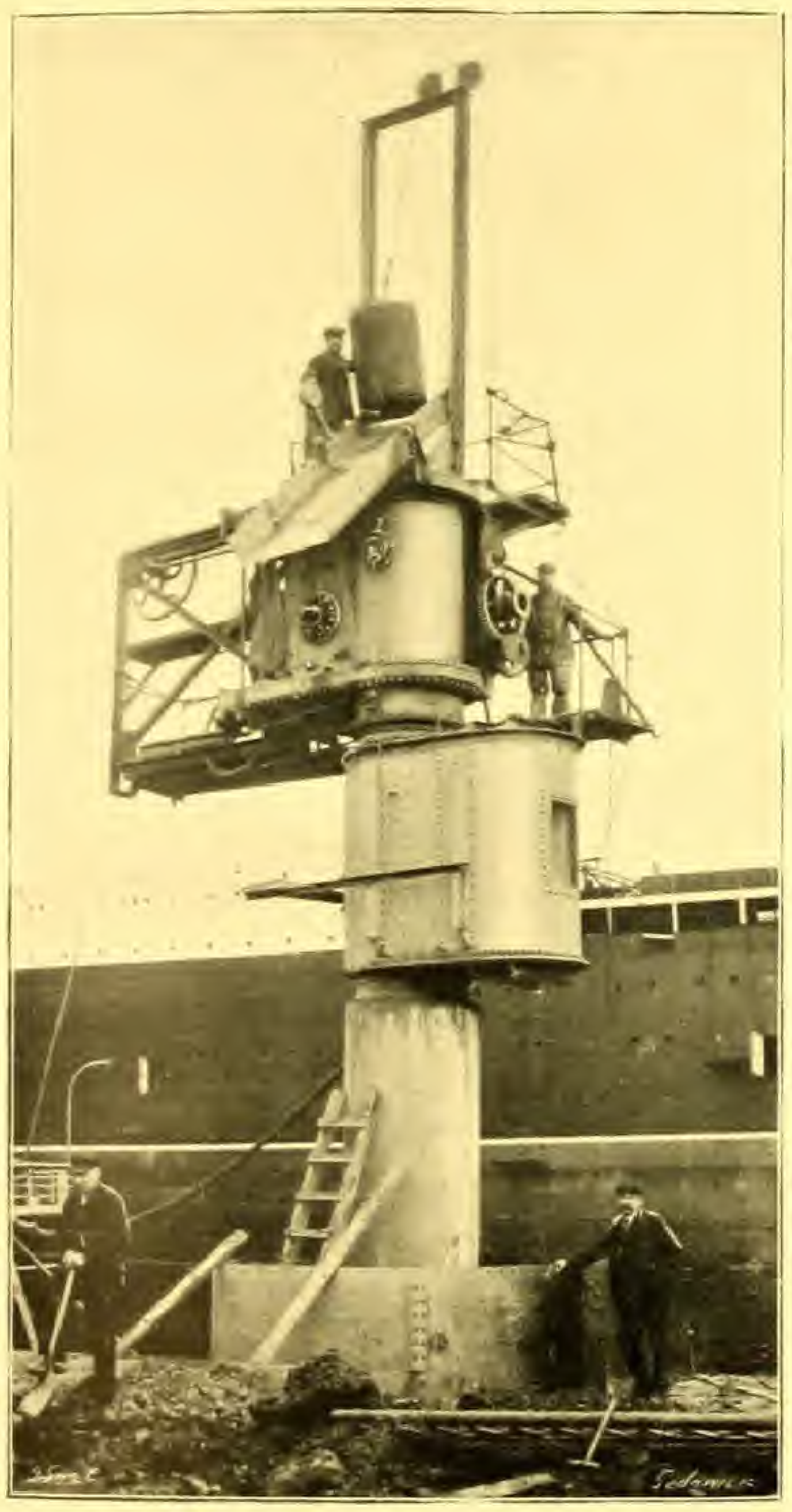
In recent years the system has been extended for almost all kinds of foundations. As illustration of such plant we give on the preceding page a view of the sinking of the Stevenston pit-shafts of the Glengarnock Iron and Steel Company.

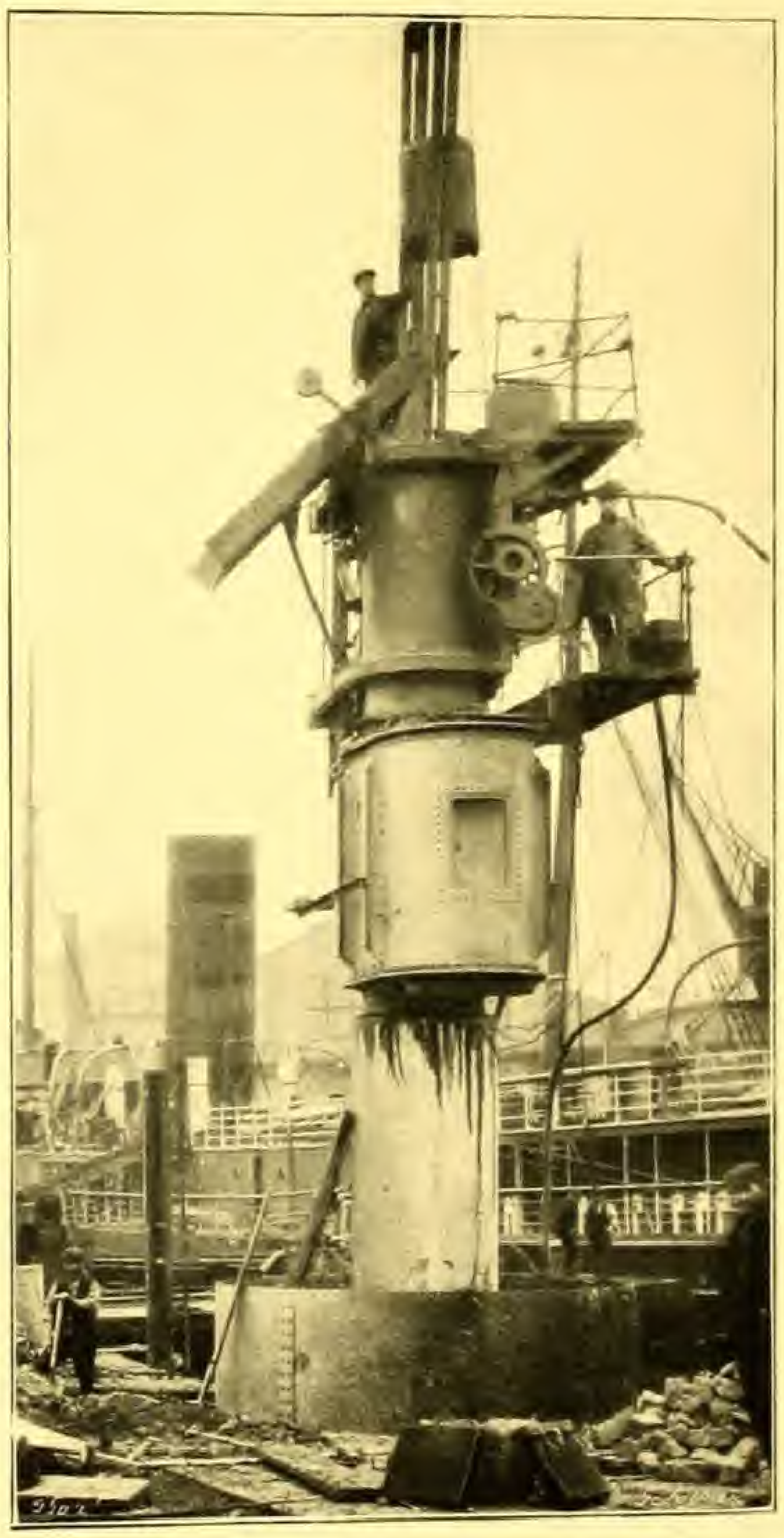
Two shafts were sunk near the foreshore through beds of sand, shells and clay, to the rock found at a depth of 84 ft. from the surface. The shaft was of steel 17 ft. 6 in. in diameter, lined with brickwork 20 in. thick during the process of sinking. Here, as always, special details had to be devised to meet unusual conditions; in this instance, a sound water-tight shaft was of great importance, in view of the proximity of the sea.

The photographs reproduced on the opposite page show the form of lock made by the firm for use in connection with air work. The lower man-lock has two compartments. It is semi-circular at the ends and flat in the centre, and has capacity for three men in each of the two compartments. The lock is built up of steel, the flat portions being stiffened with beams and angles. The doors are steel castings with rubber joints. Bulls-eye glasses are provided. All the joints are caulked, and the whole is tested to 50 lb. per square inch. There are air-cocks to enable the workmen to regulate the pressure-air when passing from one compartment to the other; the outer space is used as an intermediate stage in entrance and exit.

The material lock is placed above the man-lock. The doors in this case are horizontal, and are opened and shut by a hand rack-motion, worked from large hand wheels. There is provided a small steam engine for working the winding drum. In order to throw this lifting drum out of gear quickly, a clutch is provided, so that

when the buckets are resting on the bottom door an overhead lifting arrangement may be brought into gear, and the buckets raised above the lock to tip the excavated





Air Locks for Sinking Foundations.

material into the shoot. The only additions required to complete this installation are the boilers and air compressors, with connecting pipes and fittings ; and these also are supplied by Sir William Arrol and Company, Limited.

APPENDICES.

FORMULÆ AND DIAGRAMS

FOR THE

CALCULATION OF BEAMS.

NOTE:

(i) Expressions containing E apply only to beams and cantilevers of uniform modulus of elasticity.

(ii) Expressions containing I apply only to beams and cantilevers of constant moment of inertia.

(iii) All formulæ and diagrams relating to continuous girders, and beams fixed at one or both ends, apply only to beams of constant moment of inertia and uniform modulus of elasticity.

CONVENTION OF SIGNS AND NOTATION USED THROUGHOUT.

In all cases the origin is at the left-hand support.

1. *Diagram showing Method of Loading:*

   $+ x$ measured horizontally from left to right.

   $+ y$ measured vertically downwards.

2. *Shearing Force Diagram:*

   Start the diagram from the origin. Draw reactions vertical and upwards; applied loads vertical and downwards.

3. *Bending Moment Diagram:*

   $+$ M downwards when it causes convexity downwards.

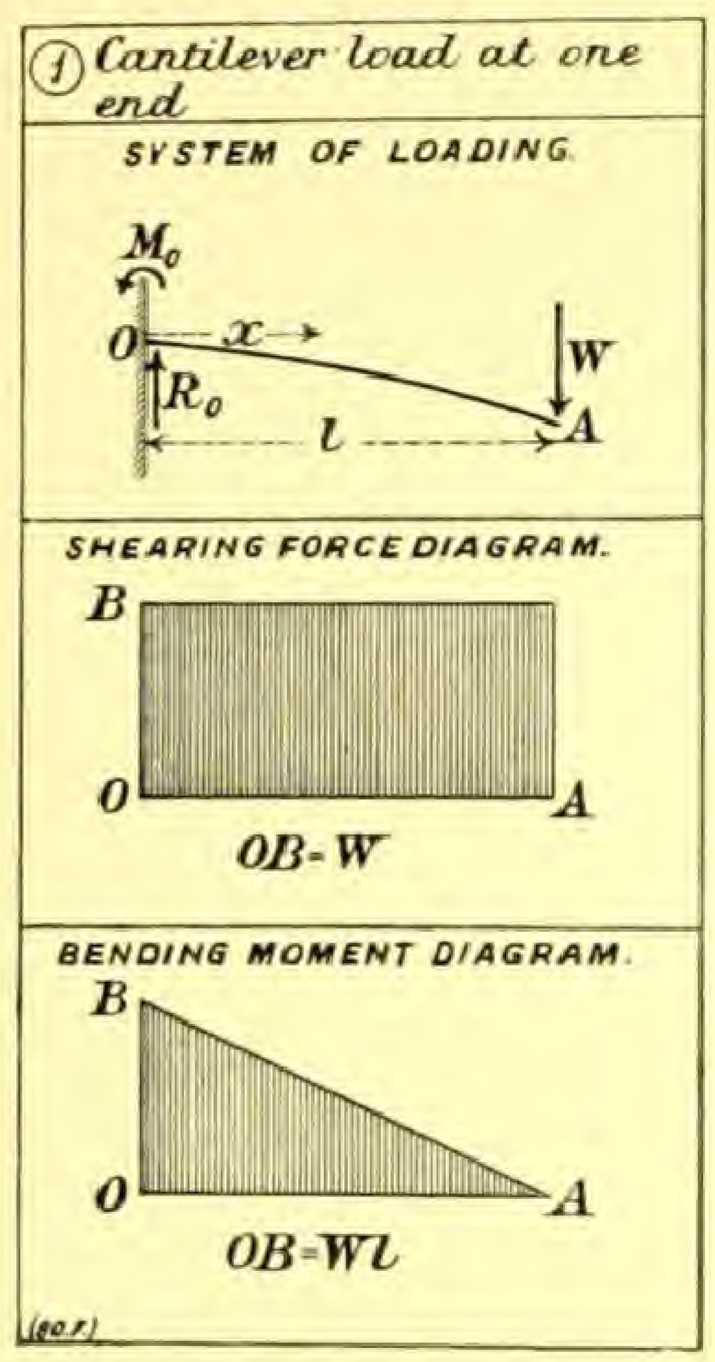
   $-$ M upwards when it causes convexity upwards.

l = Span. W = Total Load. w = Load per Foot.

M_0, M_A, M_B Bending Moments at O, A, B. R_0, R_A, R_B Reactions at O, A, B.

E = Modulus of Elasticity. I = Moment of Inertia.

Expressions involving I apply only to beams having a constant Moment of Inertia.



(1)

$$R_0 = W$$

Shearing force between O and A $= W$

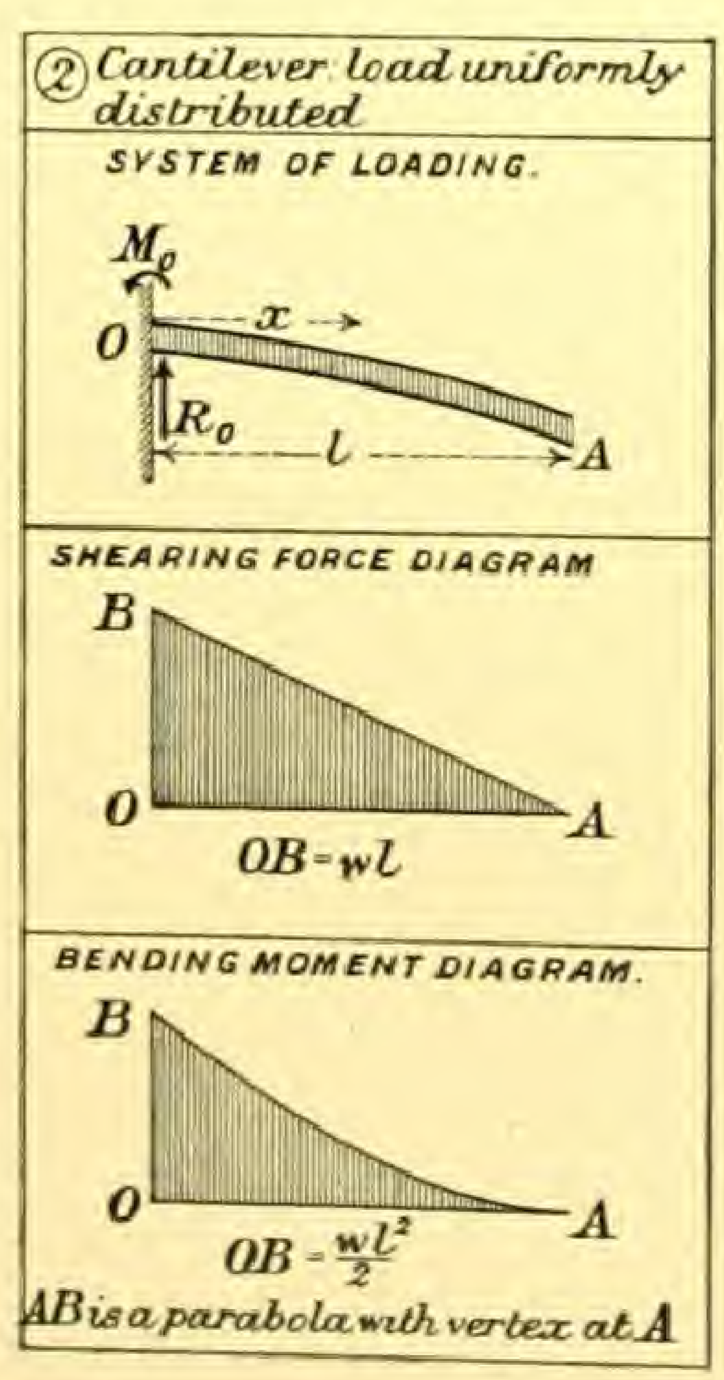
Bending moment between O and A $= -W(l - x)$

$$M_0 = -Wl$$

Equation to elastic line :

$$y = \frac{Wx^2}{6\,EI}\left(3l - x\right)$$

$$\text{Deflection at A} = \frac{Wl^3}{3\,EI}.$$



(2)

$$R_0 = wl = W$$

Shearing force between O and A $= w(l - x)$

Bending moment between O and A $= -\dfrac{w(l - x)^2}{2}$

$$M_0 = -\frac{wl^2}{2}$$

Equation to elastic line :

$$y = \frac{wx^2}{24\,EI}\left(x^2 - 4\,lx + 6\,l^2\right)$$

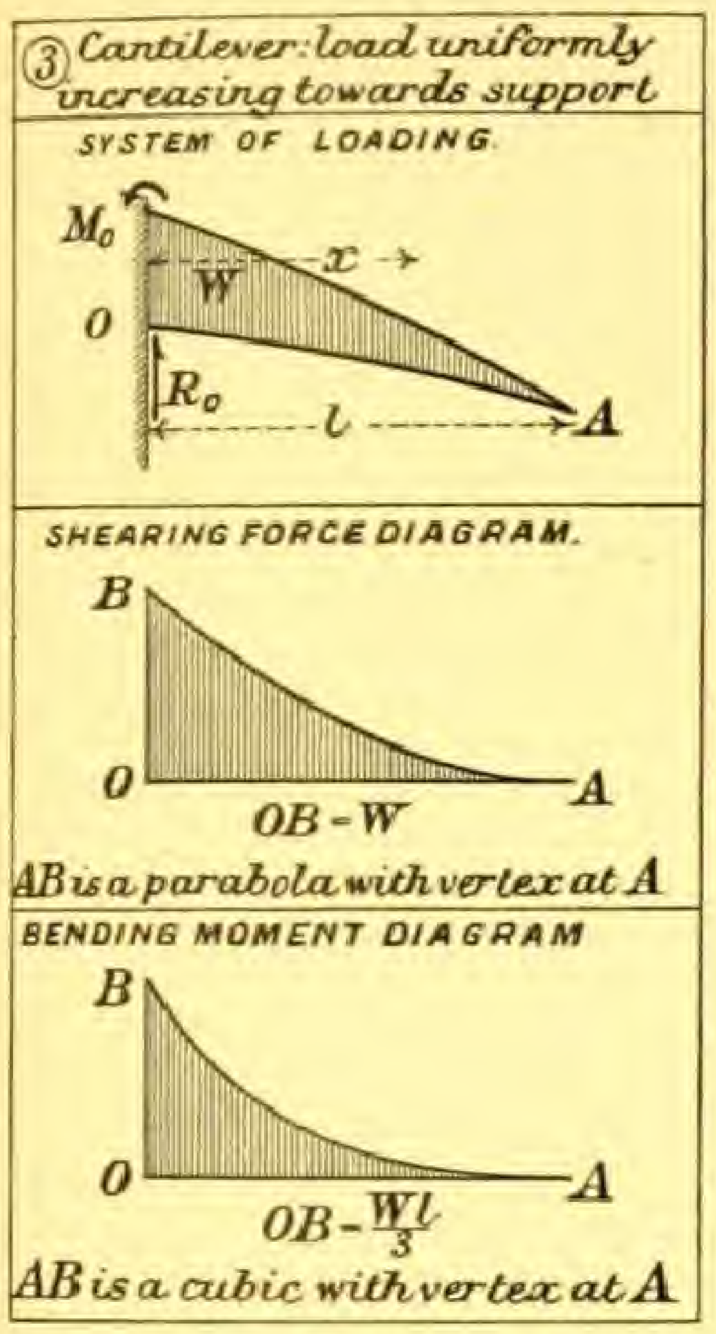
$$\text{Deflection at A} = \frac{Wl^3}{8\,EI}.$$

l = Span. W = Total Load. w = Load per Foot.

M_0, M_A, M_B Bending Moments at O, A, B. R_0, R_A, R_B Reactions at O, A, B.

E = Modulus of Elasticity. I = Moment of Inertia.

Expressions involving I apply only to beams having a constant Moment of Inertia.



(3)

$$R_0 = W = \text{Total load}$$

Intensity of load between O and A $= \dfrac{2\,W}{l^2}\left(l - x\right)$

Intensity of load at O $= \dfrac{2\,W}{l}$

Shearing force between O and A $= \dfrac{W\,(l - x)^2}{l^2}$

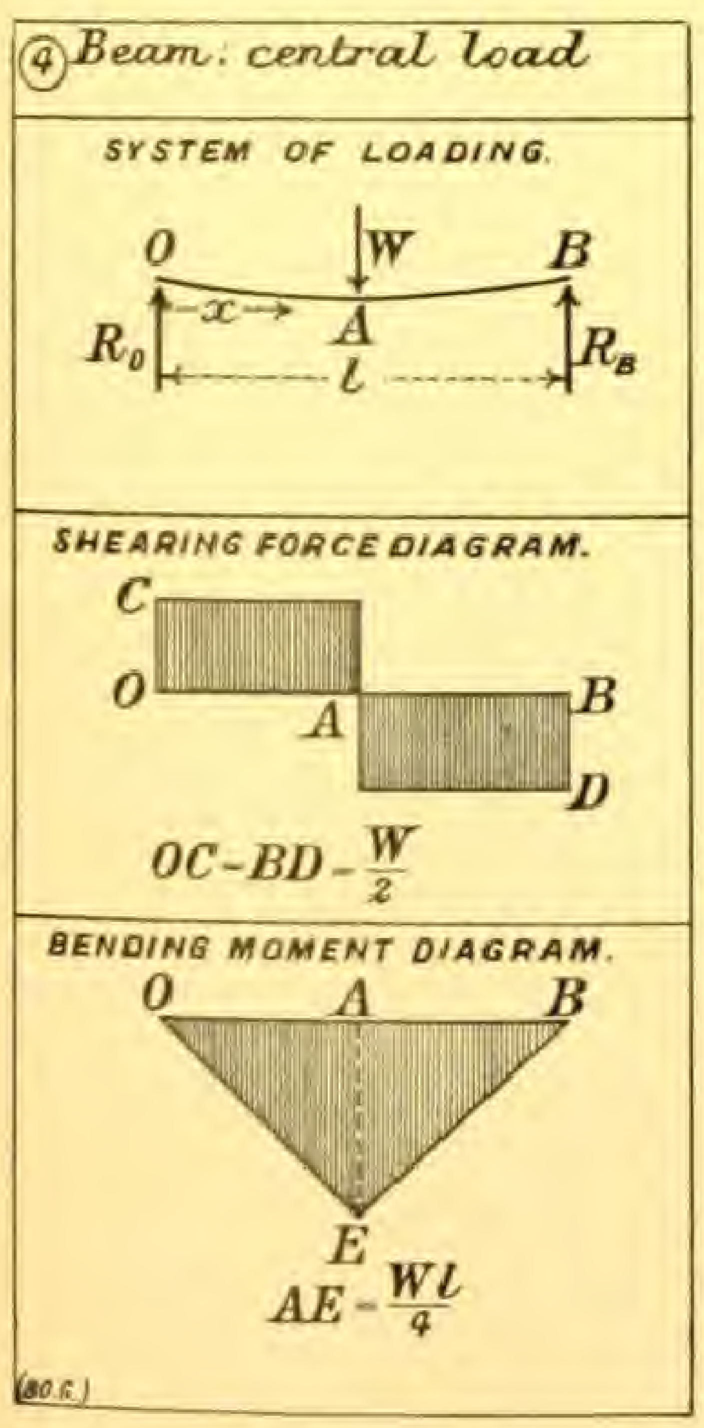
Bending moment between O and A $= -\dfrac{W\,(l - x)^3}{3\,l^2}$

$$M_0 = -\frac{Wl}{3}$$

Equation to elastic line:

$$y = \frac{Wx^2}{60\,EIl^2}\left(10\,l^3 - 10\,l^2x + 5\,lx^2 - x^3\right)$$

Deflection at A $= \dfrac{Wl^3}{15\,EI}$.



(4)

$$R_0 = R_B = \frac{W}{2}$$

Shearing force between O and A $= \dfrac{W}{2}$

Bending moment: $x \leqq \dfrac{l}{2}$

$$M_x = \frac{Wx}{2}$$

$$M_0 = M_B = 0 \qquad . \qquad M_A = \frac{Wl}{4}$$

Equation to elastic line: $x \leqq \dfrac{l}{2}$

$$y = \frac{Wx}{48\,EI}\left[3l^2 - 4x^2\right]$$

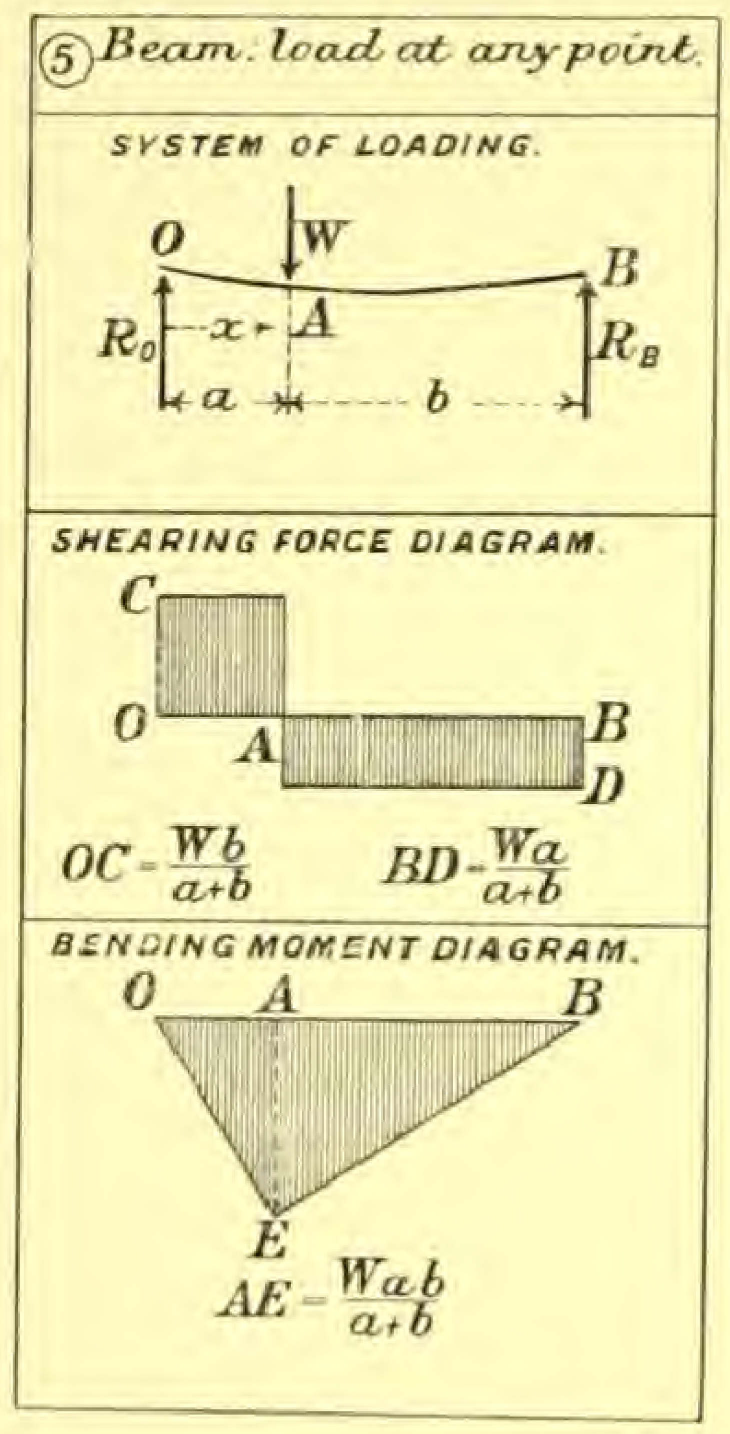
Deflection at centre $= \dfrac{Wl^3}{48\,EI}$.

l = Span. W = Total Load. w = Load per Foot.

M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.

E = Modulus of Elasticity. I = Moment of Inertia.

Expressions involving I apply only to beams having a constant Moment of Inertia.



(5) $$R_O = \frac{Wb}{a + b} \quad . \quad R_B = \frac{Wa}{a + b}$$

Shearing force between O and A = R_O

Shearing force between A and B = R_B

Bending moment:

(i) $x \leqq a$. $$M_x = \frac{Wbx}{a + b}$$

(ii) $x \geqq a$. $$M_x = \frac{Wbx}{a + b} - W(x - a)$$

$$M_O = M_B = O \quad . \quad M_A = \frac{Wab}{a + b}$$

Equations to elastic line:

(i) $x \leqq a$

$$y = \frac{Wbx}{6\,EI\,(a + b)}\left[- x^2 + a\,(a + 2\,b)\right]$$

(ii) $x \geqq a$

$$y = \frac{Wa}{6\,EI\,(a + b)}\left[(x - a)^3 - 3\,bx^2 + 2\,b^2x + 4\,abx - a^2b\right]$$

$$\text{Deflection at A} = \frac{Wa^2b^2}{3\,EI\,(a + b)}$$

Deflection at centre: $a \leqq b$ $\delta = \frac{Wa}{48\,EI}\left(3\,b^2 + 6\,ab - a^2\right)$

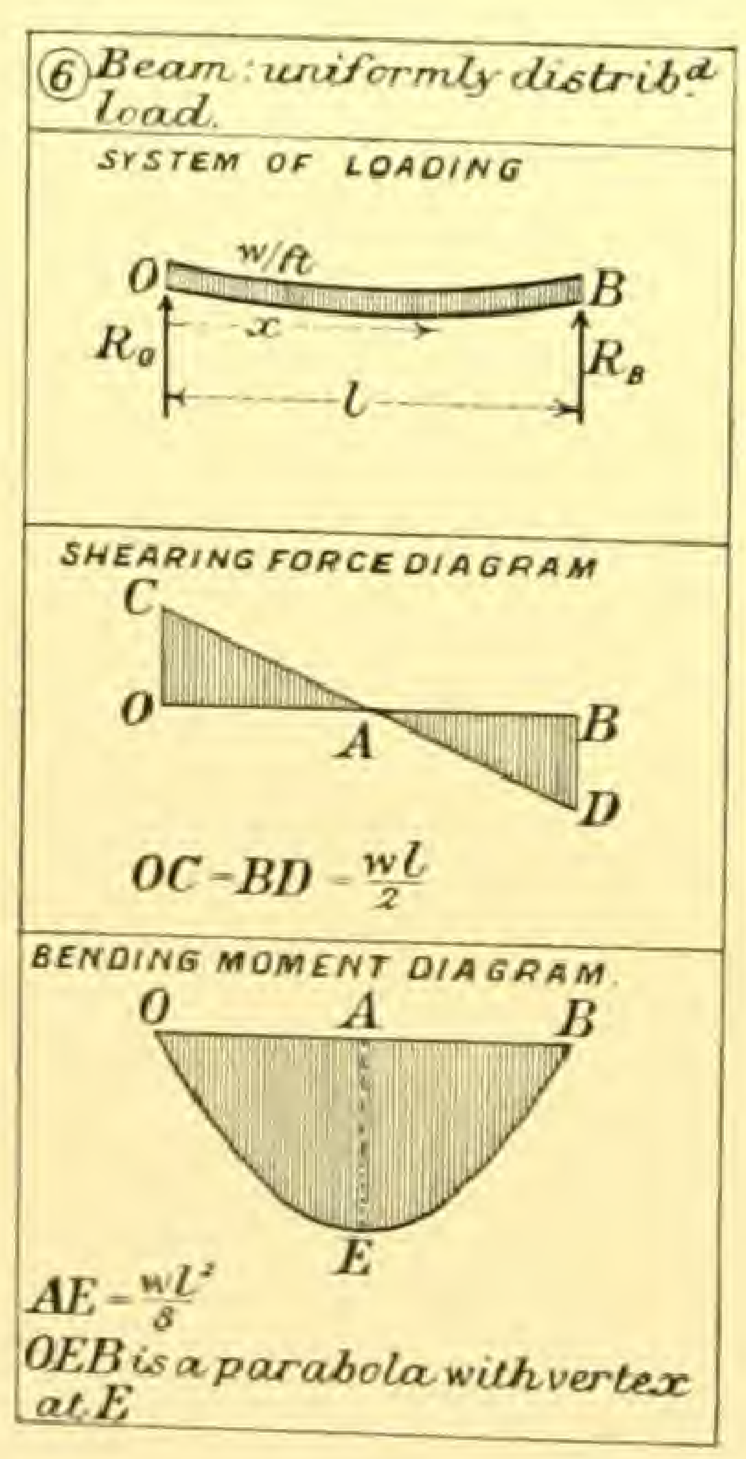
Maximum deflection:

(i) $a \geqq b$

$$\delta_{\text{max.}} = \frac{Wab\,(a + 2\,b)}{9\,EI\,(a + b)}\sqrt{\frac{a^2 + 2\,ab}{3}} \quad \text{at point where } x = \sqrt{\frac{a^2 + 2\,ab}{3}}$$

(ii) $a \leqq b$

$\delta_{\text{max.}}$ occurs at point where $x = a + b - \sqrt{\frac{b^2 + 2\,ab}{3}}$.



(6) $$R_O = R_B = \frac{wl}{2}$$

Shearing force between O and B = $\frac{w}{2}\left(l - 2\,x\right)$

Bending moment:

$$M_x = \frac{wx}{2}\left(l - x\right)$$

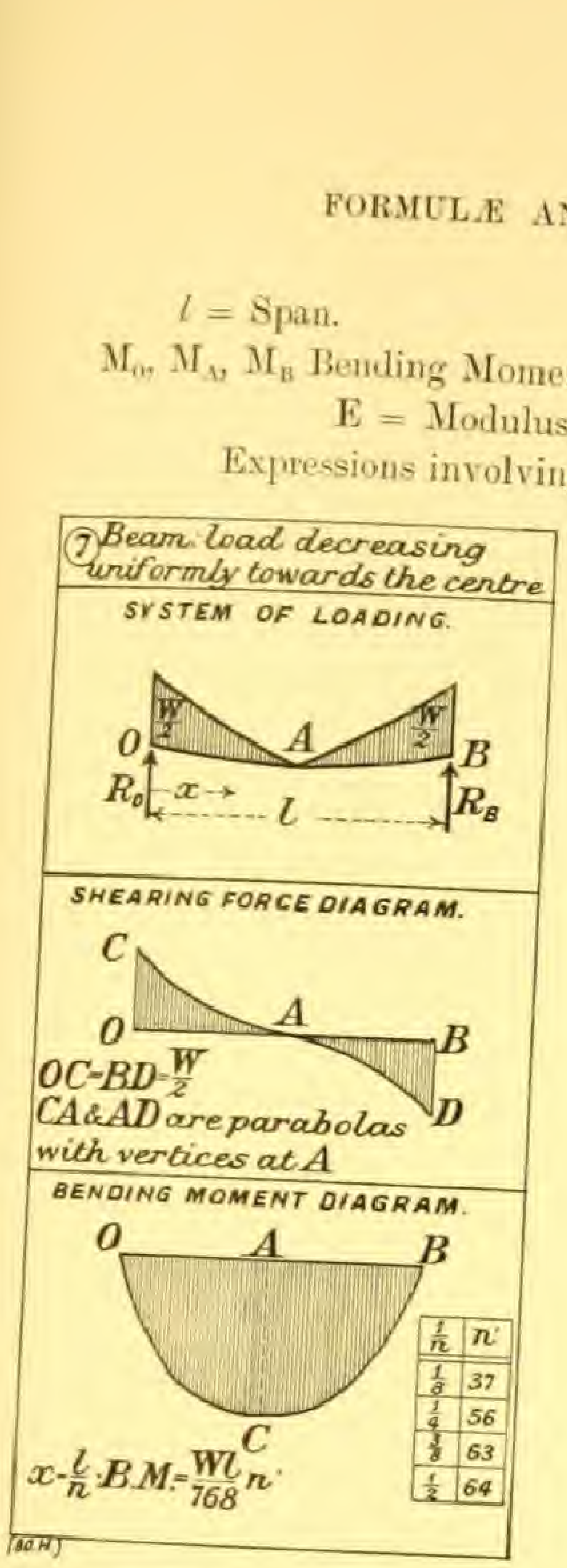
$$M_O = M_B = O \quad . \quad M_A = + \frac{wl^2}{8}$$

Equation to elastic line:

$$y = \frac{wx}{24\,EI}\left(x^3 - 2\,lx^2 + l^3\right)$$

$$\text{Deflection at centre} = \frac{5}{384}\,\frac{Wl^3}{EI}.$$

l = Span. W = Total Load. w = Load per Foot.
M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.
E = Modulus of Elasticity. I = Moment of Inertia.
Expressions involving I apply only to beams having a constant Moment of Inertia.



(7)

$$R_O = R_B = \frac{W}{2}$$

Intensity of load at O $= \dfrac{2W}{l}$

Intensity of load between O and A $= \dfrac{2W(l-2x)}{l^2}$

Shearing force between O and A $= \dfrac{W}{2l^2}\left[4x^2 - 4lx + l^2\right]$

Shearing force at O $= \dfrac{W}{2}$

Bending moment : $x \leqq \dfrac{l}{2}$

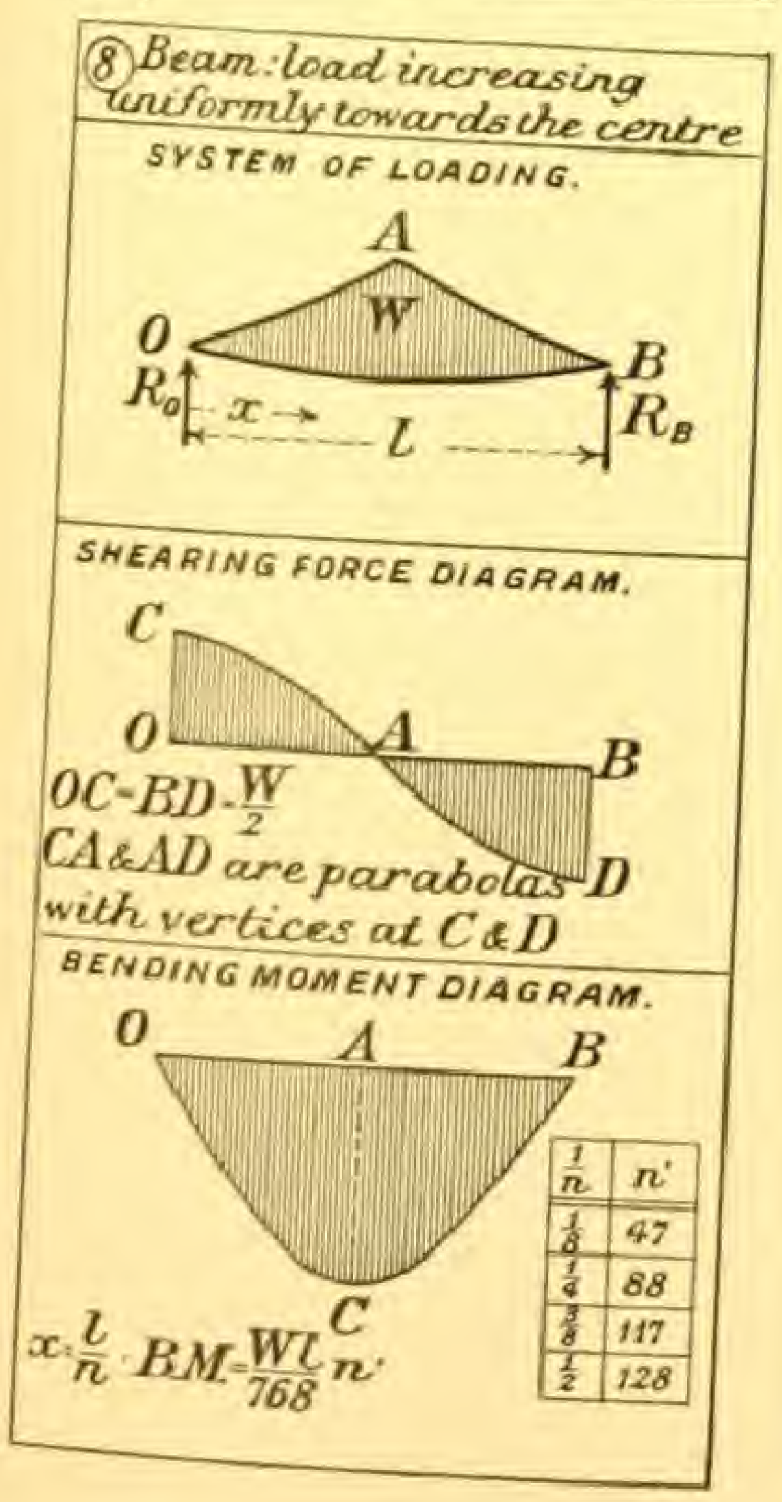
$$M_x = \frac{Wx}{6l^2}\left[4x^2 - 6lx + 3l^2\right]$$

$$M_O = M_B = 0 \quad . \quad M_A = \frac{Wl}{12}$$

Equation to elastic line : $x \leqq \dfrac{l}{2}$

$$y = \frac{Wx}{480\,EI\,l^2}\left[-16x^4 + 40lx^3 - 40l^2x^2 + 15l^4\right]$$

Deflection at centre $= \dfrac{3Wl^3}{320\,EI}$.



(8)

$$R_O = R_B = \frac{W}{2}$$

Intensity of load at A $= \dfrac{2W}{l}$

Intensity of load between O and A $= \dfrac{4Wx}{l^2}$

Shearing force between O and A $= \dfrac{W}{2l^2}\left(l^2 - 4x^2\right)$

Shearing force at O $= \dfrac{W}{2}$

Bending moment : $x \leqq \dfrac{l}{2}$

$$M_x = \frac{Wx}{6l^2}\left[3l^2 - 4x^2\right]$$

$$M_O = M_B = 0 \quad . \quad M_A = \frac{Wl}{6}$$

Equation to elastic line : $x \leqq \dfrac{l}{2}$

$$y = \frac{Wx}{480\,EIl^2}\left[16x^4 - 40l^2x^2 + 25l^4\right]$$

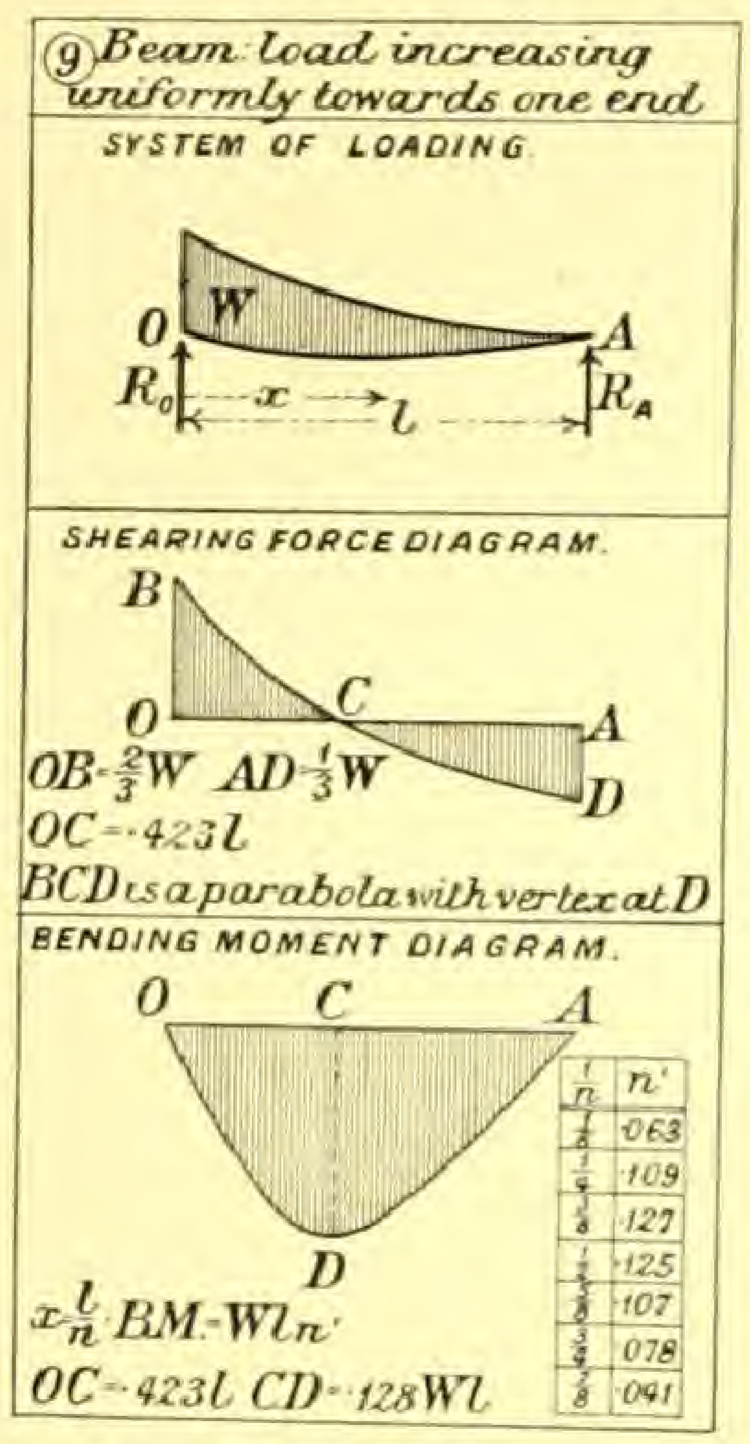
Deflection at centre $= \dfrac{Wl^3}{60\,EI}$.

l = Span. W = Total Load. w = Load per Foot

M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.

E = Modulus of Elasticity. I = Moment of Inertia.

Expressions involving I apply only to beams having a constant Moment of Inertia.



(9) Total load = W

$$\text{Intensity of load at O} = \frac{2\,W}{l}$$

$$\text{Intensity of load between O and A} = \frac{2\,W\,(l - x)}{l^2}$$

$$\text{Shearing force between O and A} = \frac{W}{3\,l^2}\left[3\,x^2 - 6\,lx + 2\,l^2\right]$$

$$\text{Shearing force at O} = \frac{2\,W}{3}$$

$$\text{Shearing force at A} = -\frac{W}{3}$$

Bending moment: $x \leq l$

$$M_x = \frac{W\,x}{3\,l^2}\left[x^2 - 3\,lx + 2\,l^2\right]$$

$$M_O = M_A = O$$

Maximum bending moment = .128 Wl

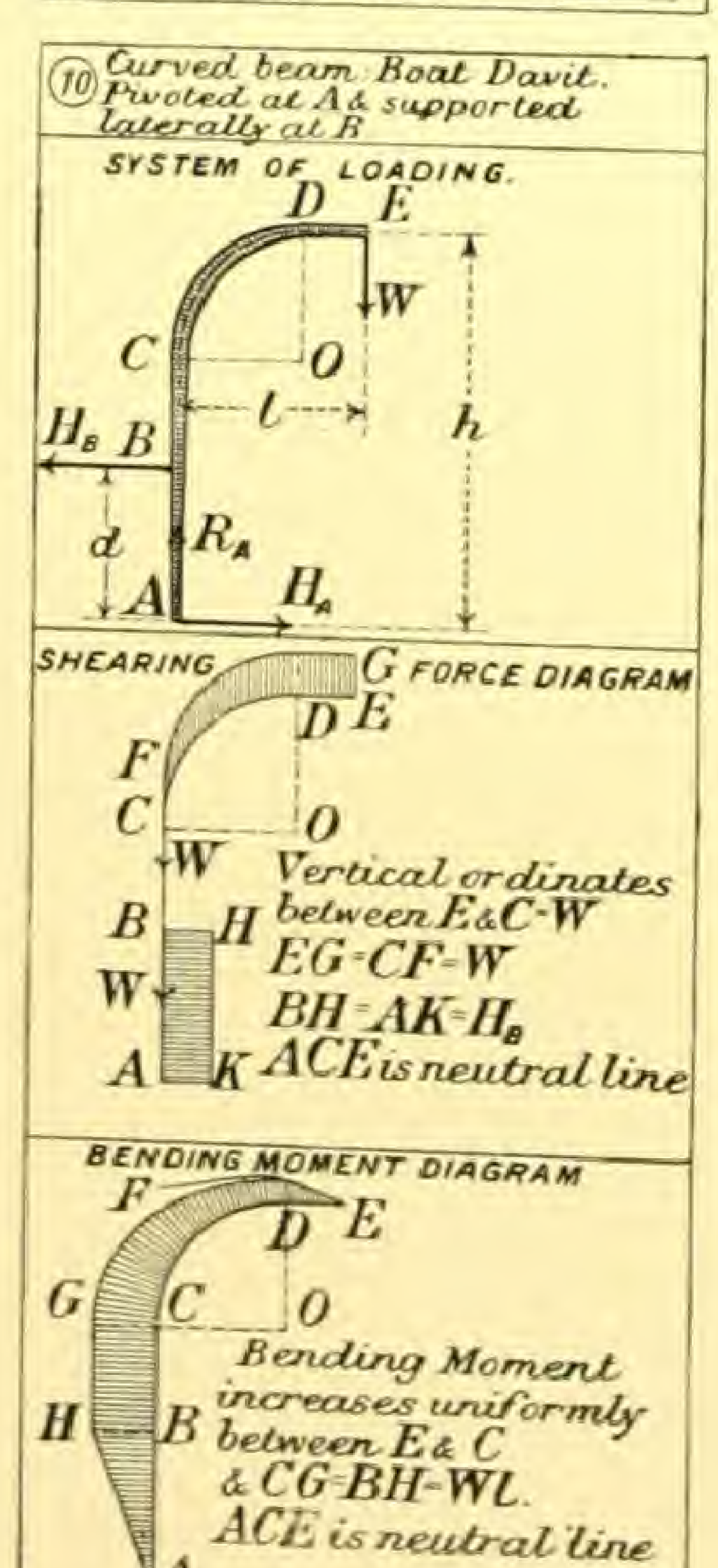
where x = .423 l

Equation to elastic line: $x \leq l$

$$y = \frac{W\,x}{180\,EIl^2}\left[-3\,x^4 + 15\,lx^3 - 20\,l^2x^2 + 8\,l^4\right]$$

$$\text{Maximum deflection} = \frac{47\,W\,l^3}{3600\,EI};$$

where x = .48 l.



(10) R_A = W $\qquad H_A = H_B = \dfrac{W\,l}{d}$

Shearing force between E and C = W

Shearing force between B and A = $H_B = H_A$

Shearing force down CA = W

$M_E = M_A = O$. $\quad M_C = M_B = W\,l$

Deflection:

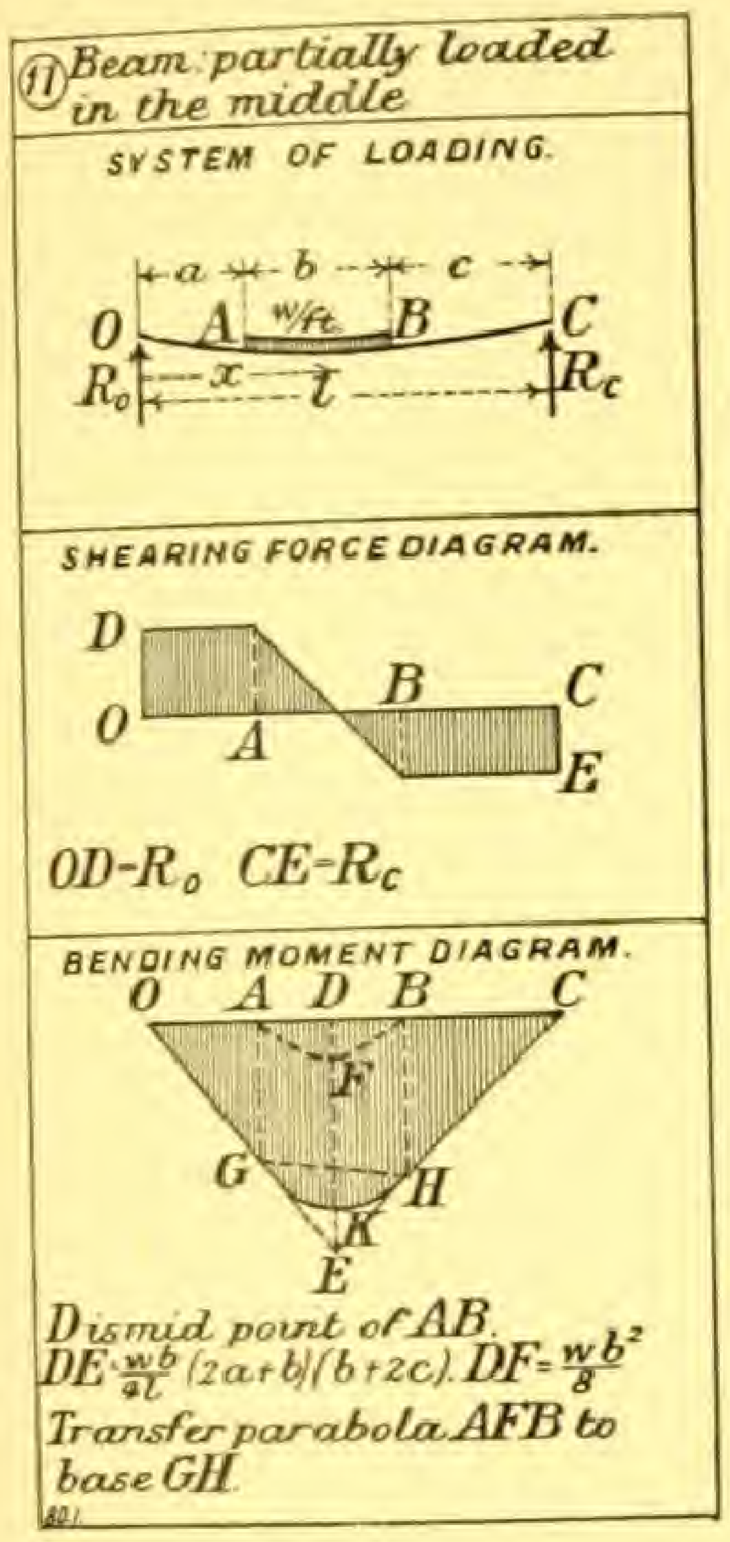
Vide "Statically Indeterminate Structures," by H. M. Martin, Wh. Sc. (page 36; 1895).

l = Span. W = Total Load. w = Load per Foot.

$\mathrm{M_0, M_A, M_B}$ Bending Moments at O, A, B. $\mathrm{R_0, R_A, R_B}$ Reactions at O, A, B.

E = Modulus of Elasticity. I = Moment of Inertia.

Expressions involving I apply only to beams having a constant Moment of Inertia.



(11) $$\mathrm{R_0} = \frac{wb}{2\,l}\left(b + 2\,c\right) \quad . \quad \mathrm{R_c} = \frac{wb}{2\,l}\left(2\,a + b\right)$$

Bending moment:

(i) $x \leqq a$

$$\mathrm{M}_x = \frac{wbx}{2\,l}\left(b + 2\,c\right)$$

(ii) $x \geqq a$ and $\leqq (a + b)$

$$\mathrm{M}_x = \frac{w}{2\,l}\left\{bx\,(b + 2\,c) - l\,(x - a)^2\right\}$$

(iii) $x \geqq (a + b)$ and $\leqq (a + b + c)$

$$\mathrm{M}_x = \frac{wb}{2\,l}\left\{(2\,a + b)\,(l - x)\right\}$$

$$\mathrm{M_A} = \frac{wab}{2\,l}\left(b + 2\,c\right) \quad . \quad \mathrm{M_B} = \frac{wbc}{2\,l}\left(2\,a + b\right)$$

$$\mathrm{M_{max.}} = \frac{wb\,(b + 2\,c)}{8\,l^2}\left\{4\,al + b\,(b + 2\,c)\right\}$$

at point where $x = \dfrac{2\,al + b\,(b + 2\,c)}{2\,l}$

Equations to elastic line:

Let $\left\{2\,a^2\,(b + 2\,c) + b^2\,(4\,a + b + 4\,c) + 4\,c^2\,(2\,a + b) + 12\,abc\right\} = m$

(i) $x \leqq a$

$$y = \frac{wbx}{24\,\mathrm{E}\mathrm{I}l}\left\{(m) - 2\,x^2\,(b + 2\,c)\right\}$$

(ii) $x \geqq a$ and $\leqq (a + b)$

$$y = \frac{w}{24\,\mathrm{E}\mathrm{I}l}\left\{l\,(x - a)^4 - 2\,bx^3\,(b + 2\,c) + bx\,(m)\right\}$$

(iii) $x \geqq (a + b)$ and $\leqq (a + b + c)$

Interchange a and c in equn. (i) for y, and measure x from c.

$$\text{Deflection at A} = \frac{wab}{24\,\mathrm{E}\mathrm{I}l}\left\{b^2\,(4a + b + 4c) + 4c^2\,(b + 2a) + 12\,abc\right\}$$

$$\text{Deflection at B} = \frac{wbc}{24\,\mathrm{E}\mathrm{I}l}\left\{b^2\,(4a + b + 4c) + 4a^2\,(b + 2c) + 12\,abc\right\}$$

Maximum deflection: when it occurs for

(i) $x \leqq a$

$$\delta = \frac{wbm}{36\,\mathrm{E}\mathrm{I}l}\sqrt{\frac{m}{6\,(b + 2\,c)}} \text{ at point where } x = \sqrt{\frac{m}{6\,(b + 2\,c)}}$$

(ii) $x \geqq a$ and $\leqq (a + b)$

x is found from $4\,l\,(x - a)^3 - 6\,bx^2\,(b + 2\,c) + mb = 0$ and

δ by substituting x in equation (ii) for y.

(iii) $x \geqq (a + b)$

$$\delta = \frac{wbm^1}{36\,\mathrm{E}\mathrm{I}l}\sqrt{\frac{m^1}{6\,(b + 2\,a)}} \text{ at point where } x = l - \sqrt{\frac{m^1}{6\,(b + 2\,a)}}$$

Note.—$m^1 = m$ with a and c interchanged.

l = Span. W = Total Load. w = Load per Foot.
M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.
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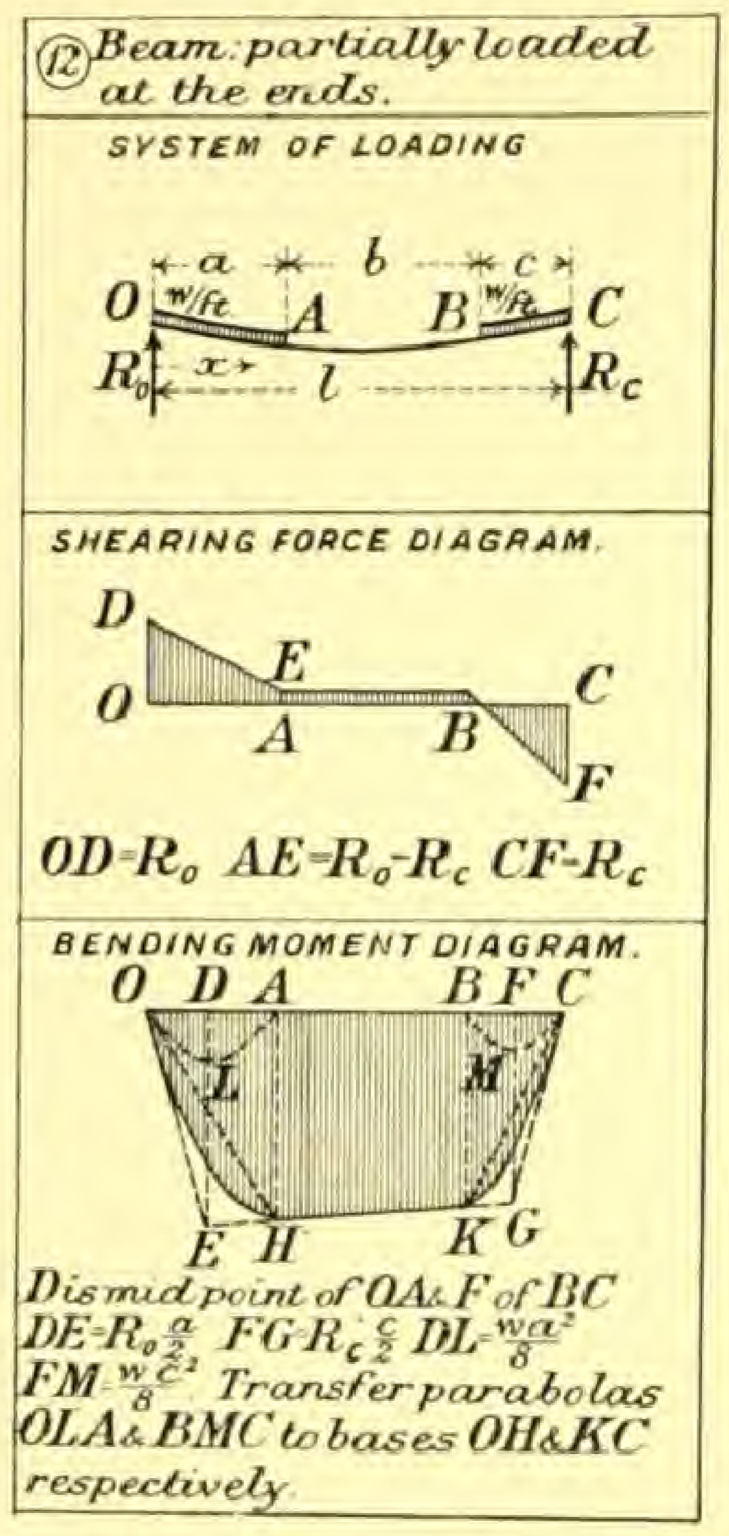


(12)

$$R_o = \frac{w}{2l}\left(2\,al - a^2 + c^2\right) \quad . \quad R_c = \frac{w}{2l}\left(2\,cl - c^2 + a^2\right)$$

Bending moment:

(i) $x \leqq a$

$$M_x = \frac{wx}{2l}\left(2\,al - a^2 + c^2 - lx\right)$$

(ii) $x \geqq a$ and $\leqq (a + b)$

$$M_x = \frac{w}{2l}\left\{x\,(c^2 - a^2) + a^2 l\right\}$$

(iii) $x \geqq (a + b)$ and $\leqq (a + b + c)$

$$M_x = \frac{w}{2l}\left\{x\,(c^2 - a^2 + 2\,al) - l\,(x - a - b)^2\right\}$$

$$M_A = \frac{wa}{2l}\left(ab + ac + c^2\right) \qquad M_B = \frac{wc}{2l}\left(ac + bc + a^2\right)$$

Equations to elastic line:

$$\text{Let } \left\{a^4 + c^4 + 4\,a^3\,(b + c) + 4\,c^3\,(a + b)\right\} = m$$

$$\left\{4\,a^2 b^2 + 2\,b^2c^2 + 6\,c^2a^2 + 4\,abc\,(2\,a + c)\right\} = n$$

(i) $x \leqq a$

$$y = \frac{wx}{24\,\mathrm{E}\mathrm{I}l}\left[lx^3 + 2\,x^2\,(a^2 - 2\,al - c^2) + (m + n)\right]$$

(ii) ($x \geqq a$ and $\leqq (a + b)$

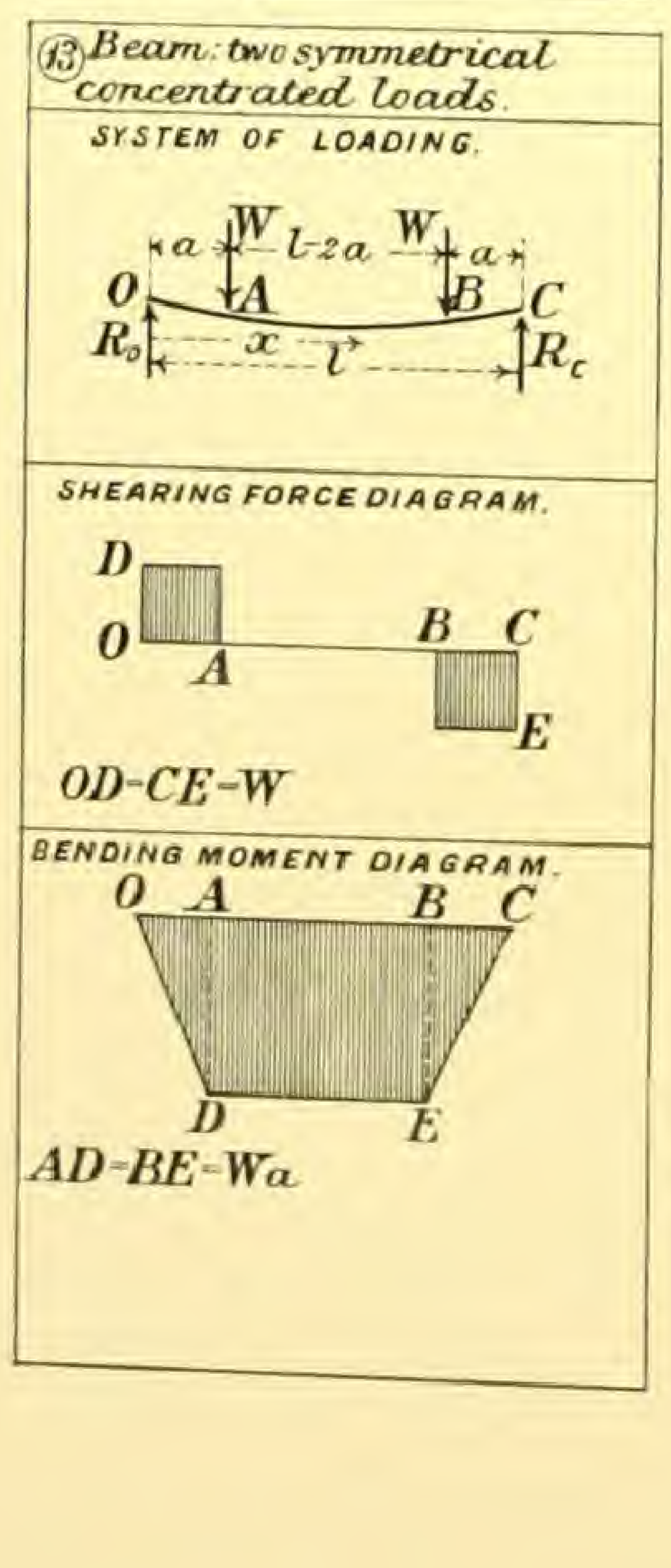
$$y = \frac{w}{24\,\mathrm{E}\mathrm{I}l}\left[2\,x^3\,(a^2 - c^2) - 6\,a^2lx^2 + x\left\{4\,a^3l + (m + n)\right\} - a^4l\right]$$

$$\text{Deflection at A} = \frac{wa}{24\,\mathrm{E}\mathrm{I}l}\left\{(m + n) - a^2\,(a^2 + 2\,c^2) - 3\,a^3\,(b + c)\right\}$$

$$\text{Deflection at B} = \frac{wc}{24\,\mathrm{E}\mathrm{I}l}\left\{(m + n) - c^2\,(c^2 + 2\,a^2) - 3\,c^3\,(b + a)\right\}$$

(iii) $x \geqq (a + b)$ and $\leqq (a + b + c)$

Interchange a and c in equn. (i) for y, and measure x from c.



(13)

$$R_o = R_c = W$$

Bending moment between O and A = Wx

Bending moment between A and B = Wa

Equations to elastic line:

$$x \leqq a \qquad y = \frac{Wx}{6\,\mathrm{EI}}\left(-x^2 - 3\,a^2 + 3\,al\right)$$

$$x \begin{matrix} > a \\ < (l - a) \end{matrix} \qquad y = \frac{Wa}{6\,\mathrm{EI}}\left(-3x^2 + 3\,lx - a^2\right)$$

$$\text{Deflection at centre} = \frac{Wa}{24\,\mathrm{EI}}\left(3\,l^2 - 4\,a^2\right)$$

$$\text{Deflection at A or B} = \frac{Wa^2}{6\,\mathrm{EI}}\left(3\,l - 4\,a\right).$$

l = Span. W = Total Load. w = Load per Foot.

M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.

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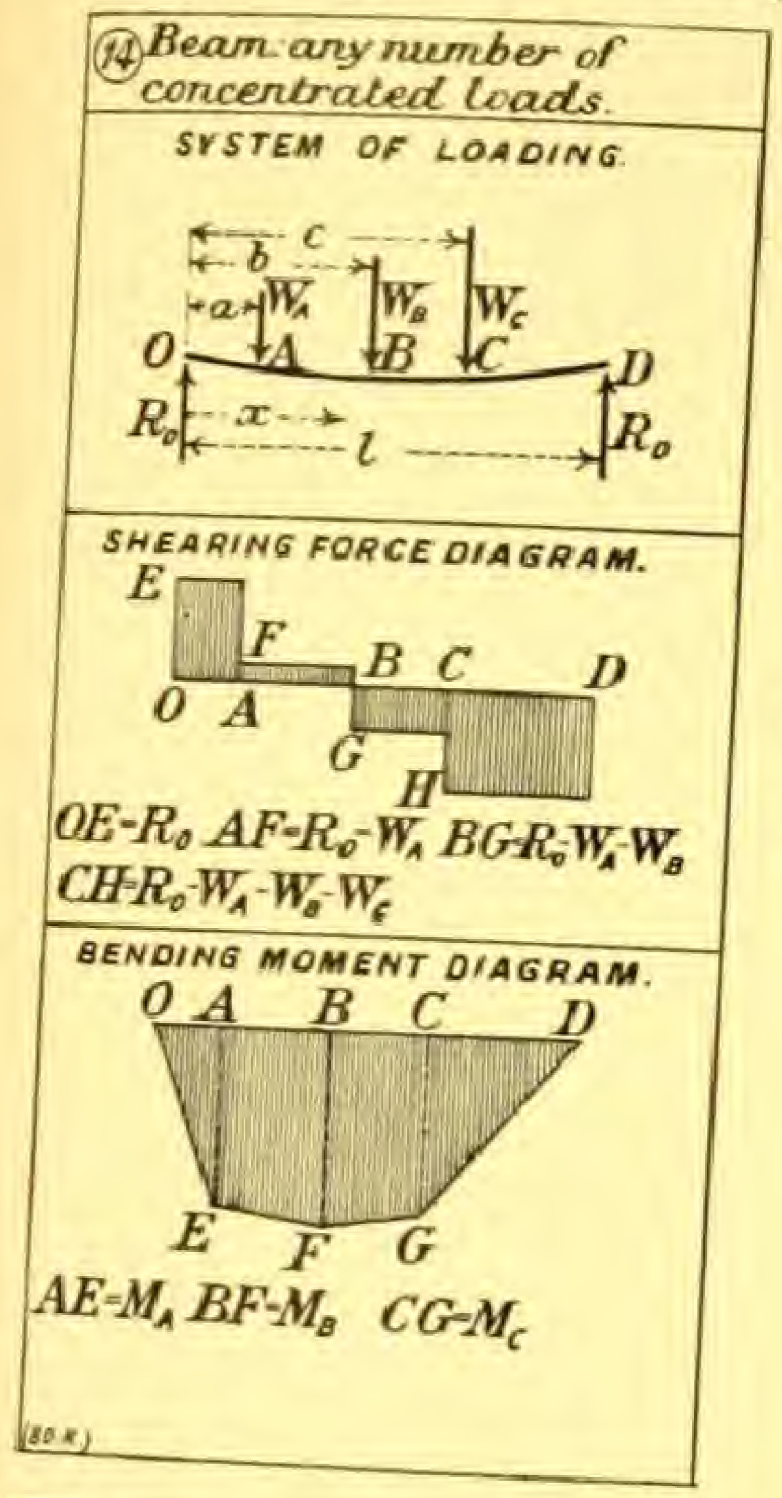


(14)

$$R_O = W_A \frac{l-a}{l} + W_B \frac{l-b}{l} + W_C \frac{l-c}{l} + \text{etc.}$$

$$R_D = W_A \frac{a}{l} + W_B \frac{b}{l} + W_C \frac{c}{l} + \text{etc.}$$

$$M_O = M_D = 0$$

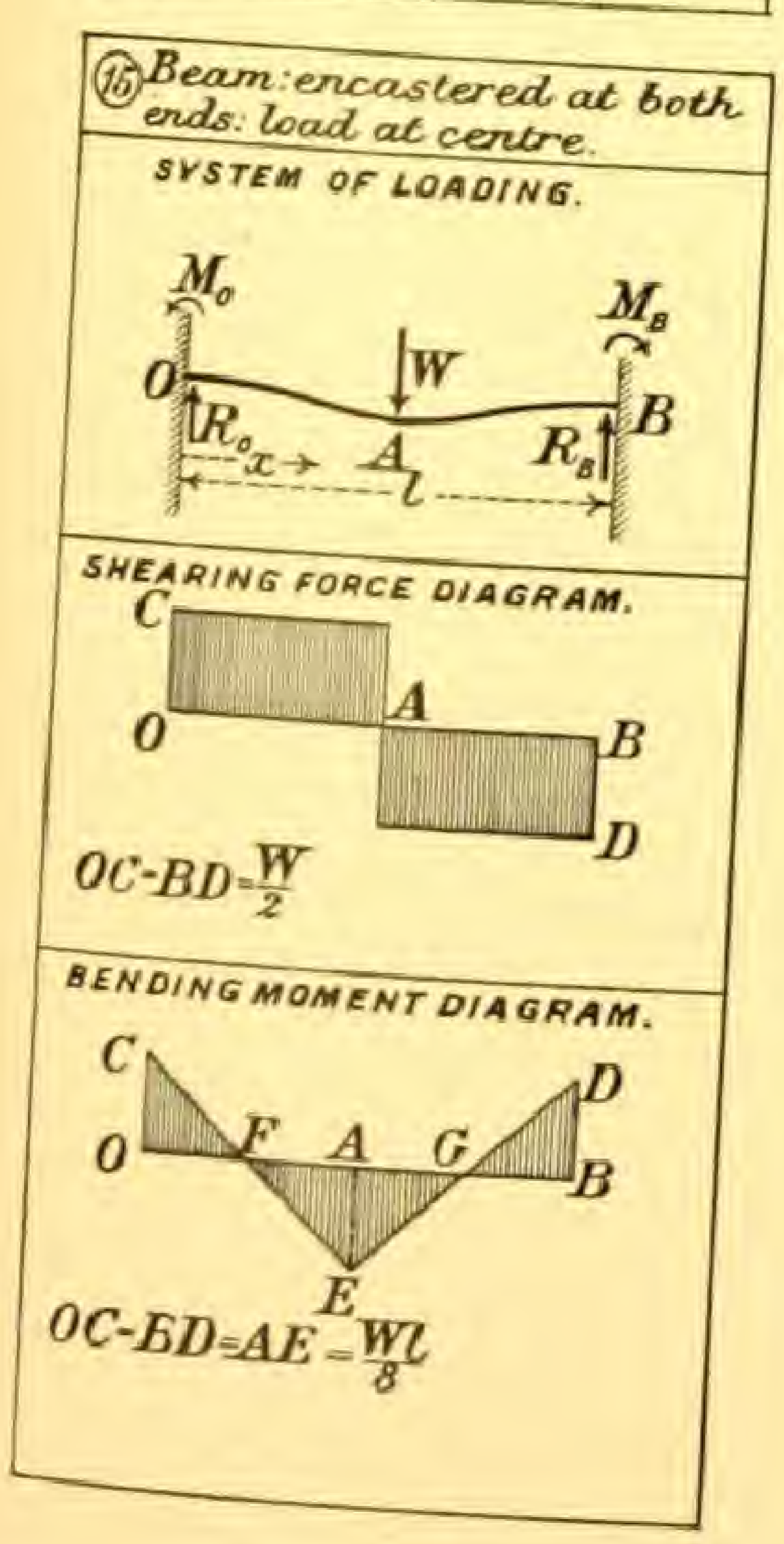
$$M_A = R_O a$$

$$M_B = R_O b - W_A (b - a)$$

$$M_C = R_O c - W_A (c - a) - W_B (c - b)$$

and so on.

Deflection at any point may be found by applying Case 5 for each load separately, and adding the partial deflections; or a graphical construction may be used. (*Vide* Goodman's "Mechanics Applied to Engineering," page 437, Fourth Edition.)



(15)

$$R_O = R_B = \frac{W}{2}$$

Bending moment: $x \leqq \frac{l}{2}$

$$M_x = \frac{W}{8}\left(4x - l\right)$$

$$M_O = M_B = -\frac{Wl}{8} \qquad . \qquad M_A = +\frac{Wl}{8}$$

Equation to elastic line: $x \leqq \frac{l}{2}$

$$y = \frac{Wx^2}{48\,EI}\left(3l - 4x\right)$$

Deflection at centre $= \frac{Wl^3}{192\,EI}$

Points of inflection:

$$OF = \frac{l}{4} \qquad . \qquad OG = \frac{3l}{4}.$$

l = Span. W = Total Load. w = Load per Foot.

M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.

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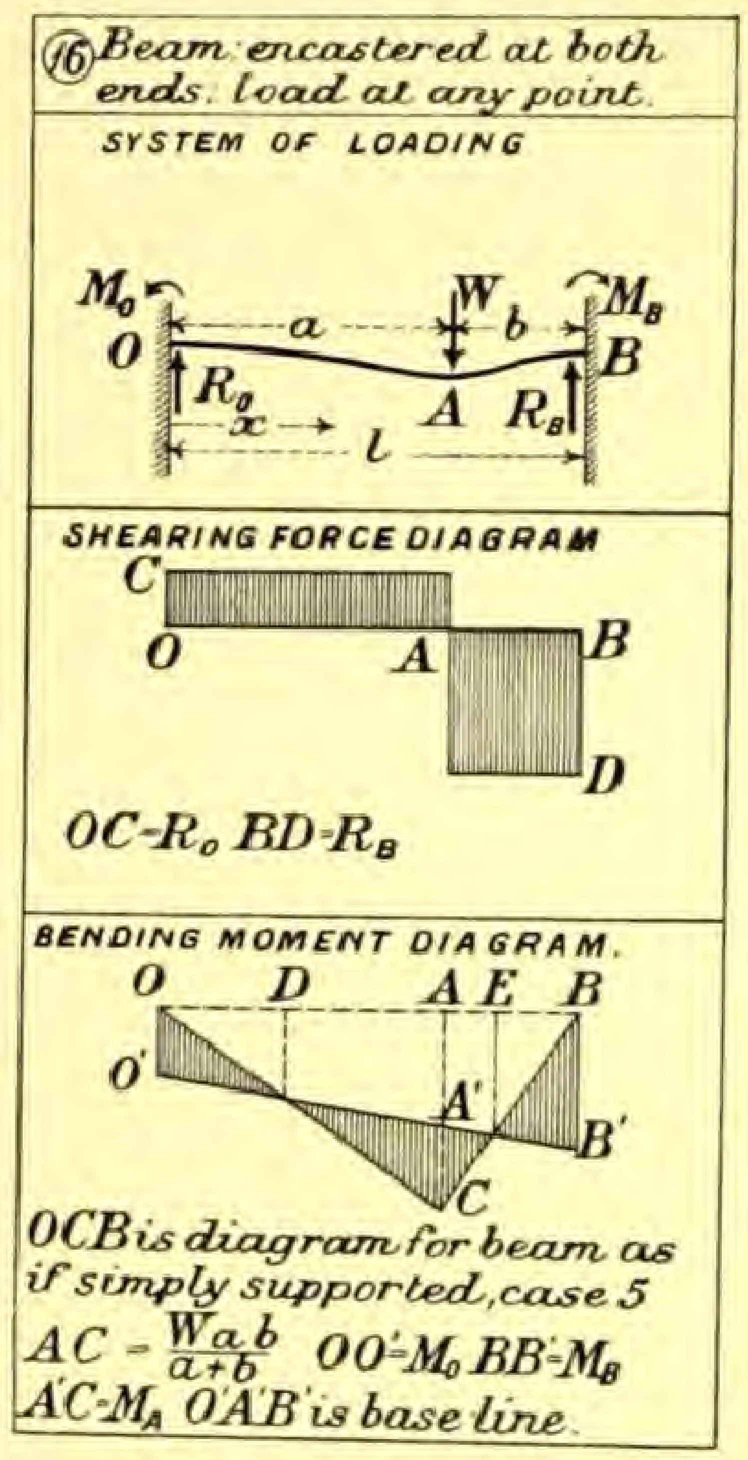

(16)

$$R_O = \frac{W}{l^3}\left(2a^3 - 3a^2 l + l^3\right)$$

$$R_B = \frac{Wa^2}{l^3}\left(3l - 2a\right)$$

$$M_O = -\frac{Wa}{l^2}\left(l - a\right)^2$$

$$M_B = -\frac{Wa^2}{l^2}\left(l - a\right)$$

$$M_A = +2\frac{Wa^2}{l^3}\left(l - a\right)^2$$

Bending moment :

(i) $x \leqq a$

$$M_x = \frac{W}{l^3}\left[x\,(2a^3 - 3a^2 l + l^3) - a\,l\,(a - l)^2\right]$$

(ii) $x \geqq a$

$$M_x = \frac{Wa^2}{l^3}\left[x\,(2a - 3l) - l\,(a - 2l)\right]$$

Equations to elastic line :

(i) $x \leqq a$

$$y = \frac{Wx^2}{6\,EIl^3}\left[-x\,(2a^3 - 3a^2 l + l^3) + 3\,al\,(a - l)^2\right]$$

(ii) $x \geqq a$

$$y = \frac{Wa^2}{6\,EIl^3}\left[-a\,(2x^3 - 3x^2 l + l^3) + 3\,xl\,(x - l)^2\right]$$

Deflection at A :

$$= \frac{Wa^3}{3\,EIl^3}\,(l - a)^3$$

Maximum deflection :

(i) $a \leqq \frac{l}{2}$

$$\delta = \frac{2\,Wa^2\,(l - a)^3}{3\,EI\,(3l - 2a)^2}, \text{ at point where } x = \frac{l^2}{3l - 2a}$$

(ii) $a \geqq \frac{l}{2}$

$$\delta = \frac{2\,Wa^3\,(a - l)^6}{3\,EI\,(2a^3 - 3a^2 l + l^3)^2}, \text{ at point where } x = \frac{2\,al\,(a - l)^2}{2a^3 - 3a^2 l + l^3}$$

Points of inflection :

$$OD = \frac{al}{2a + l} \quad ; \quad OE = \frac{l\,(2l - a)}{3l - 2a}.$$

l = Span. W = Total Load. w = Load per Foot.
M_O, M_A, M_0 Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.
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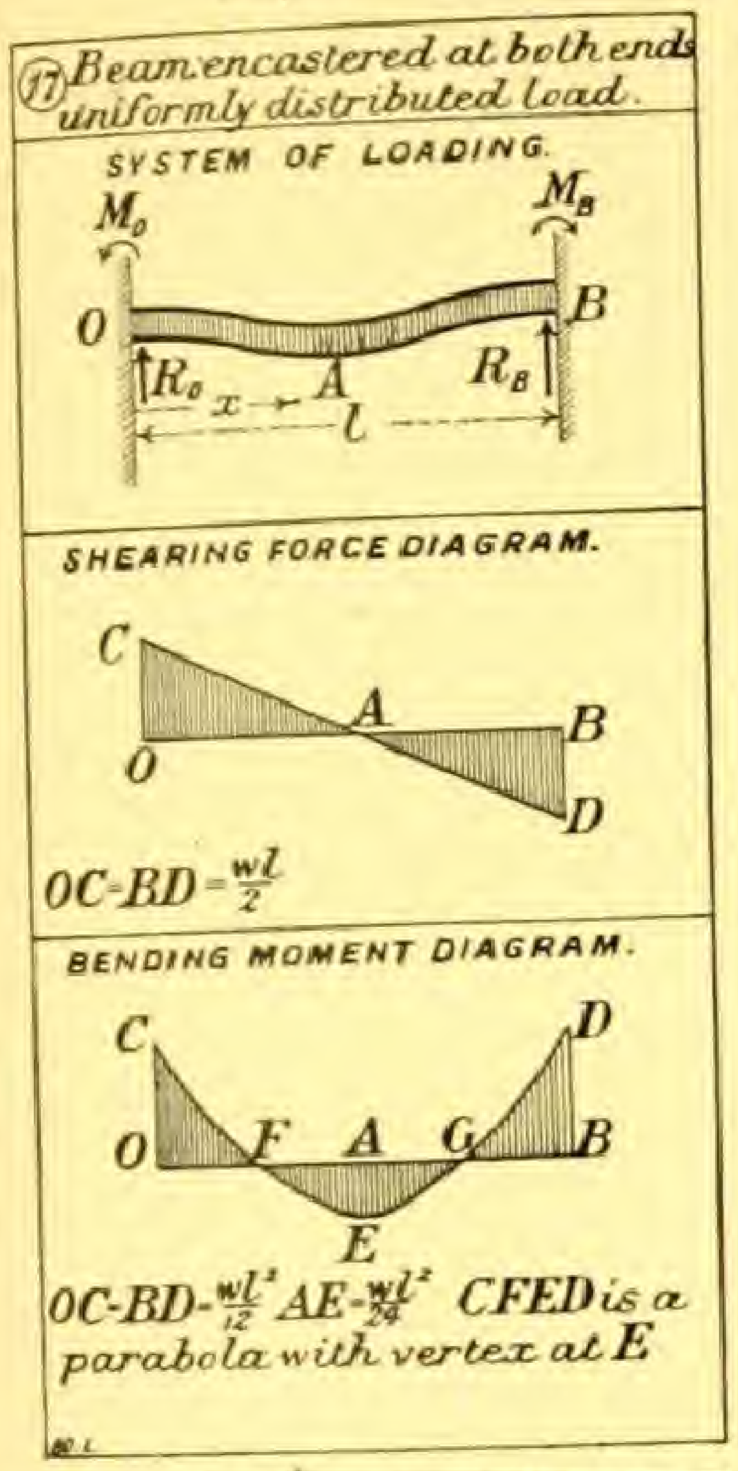
Expressions involving I apply only to beams having a constant Moment of Inertia.



(17) $$R_O = R_A = \frac{wl}{2}$$

Shearing force at any point $= \frac{w}{2}\left(l - 2x\right)$

Bending moment :

$$M_x = -\frac{w}{12}\left(6x^2 - 6lx + l^2\right)$$

$$M_O = M_B = -\frac{wl^2}{12} \quad . \quad M_A = +\frac{wl^2}{24}$$

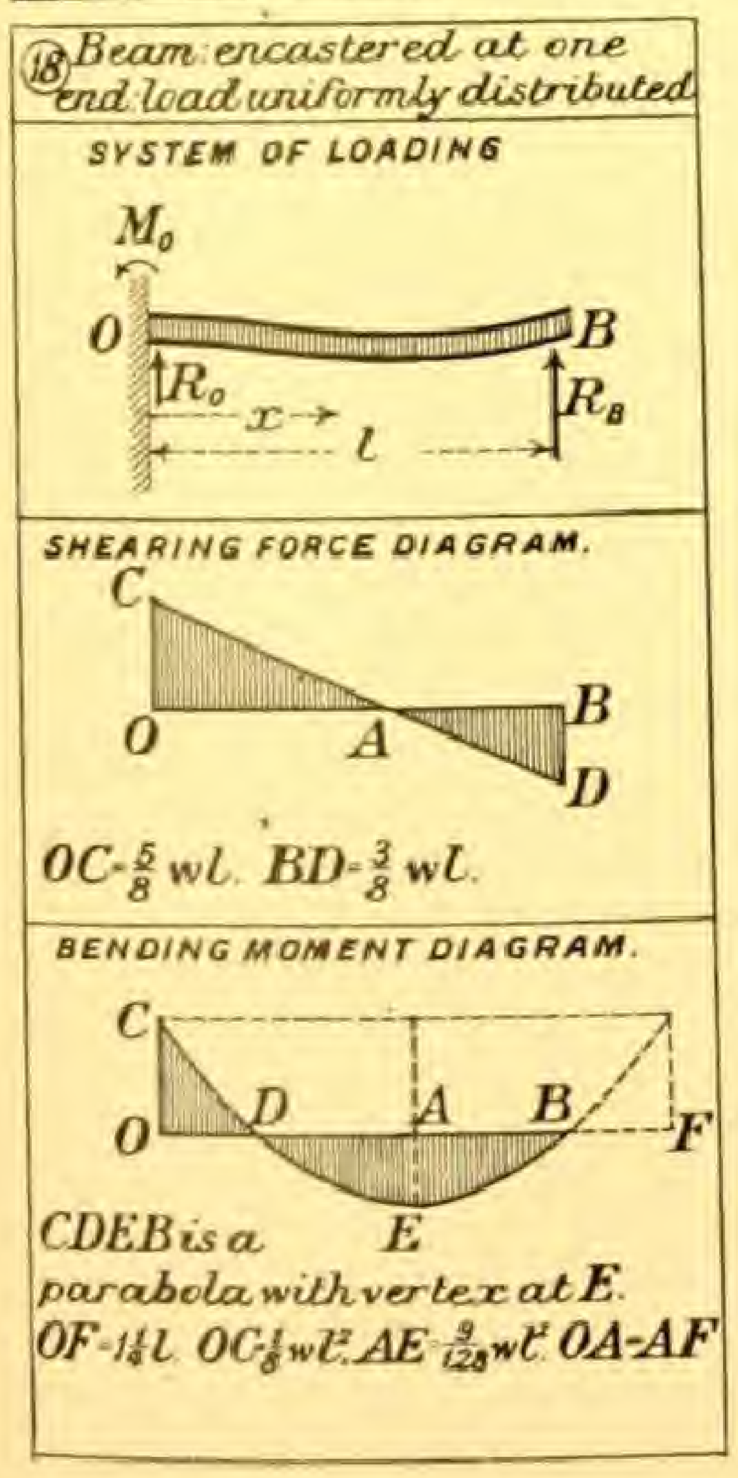
Equation to elastic line :

$$y = \frac{wx^2}{24\,EI}\left(x - l\right)^2$$

Deflection at centre $= \frac{wl^4}{384\,EI}$

Points of inflection :

OF = .211 l ; OG = .789 l.



(18) $$R_O = \frac{5}{8}\,wl \quad . \quad R_B = \frac{3}{8}\,wl$$

Shear at any point $= \frac{w}{8}\left(5l - 8x\right)$

Bending moment :

$$M_x = -\frac{w}{8}\left(4x^2 - 5lx + l^2\right)$$

$$M_O = -\frac{1}{8}\,wl^2 \quad . \quad M_B = O \quad . \quad M_A = +\frac{9}{128}\,wl^2$$

Equation to elastic line :

$$y = \frac{wx^2}{48\,EI}\left(2x^2 - 5lx + 3l^2\right)$$

Deflection at centre $= \frac{wl^3}{192\,EI}$

Maximum deflection $= 0.0054\,\frac{Wl^3}{EI}$ where $x = 0.6127\,l$

Point of inflection : OD $= \frac{l}{4}$.

l = Span. W = Total Load. w = Load per Foot.

M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.

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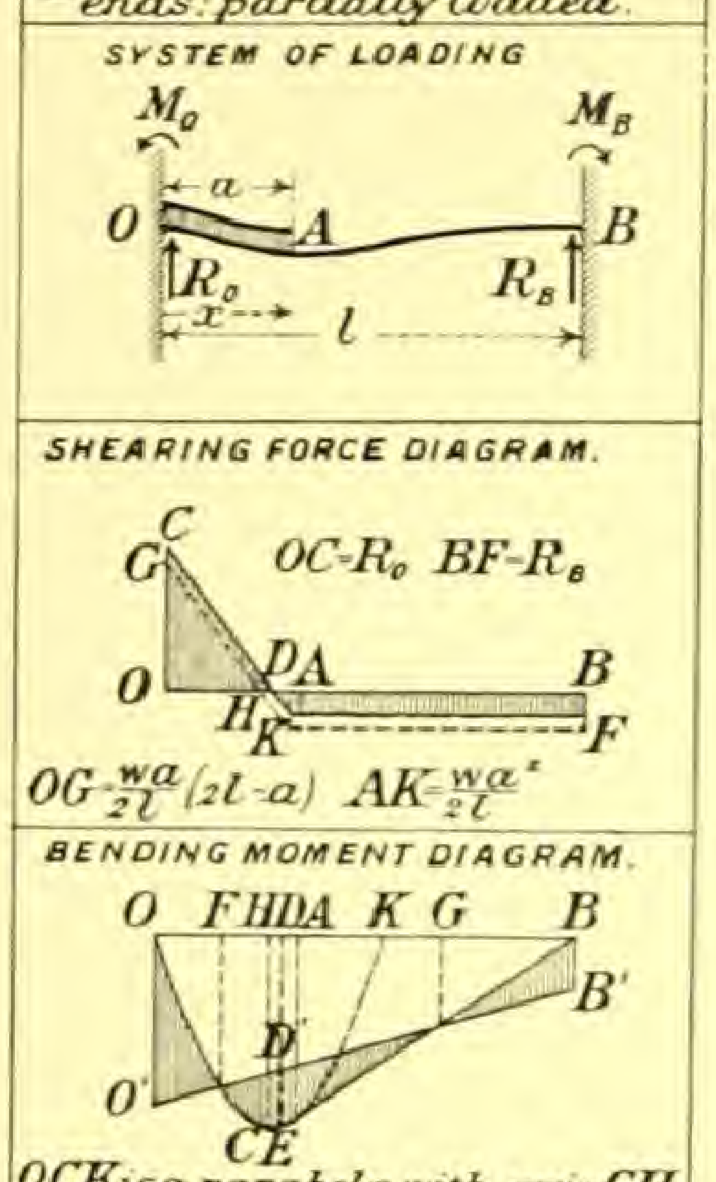
Expressions involving I apply only to beams having a constant Moment of Inertia.

(19) *Beam: encastered at both ends: partially loaded.*



OCK is a parabola with axis CH. $OH = \frac{a(2l-a)}{2l}$ $HC = \frac{wa^2}{8l^2}(2l-a)^2$ D is point of zero shearing force. $OO' = M_O$ $BB' = M_B$ $O'D'B'$ is base line

(19) NOTE.—In these formulæ,

$$m = a^3 - 2a^2l + 2l^3\ ;\ n = 3a^2 - 8al + 6l^2$$

$$p = \sqrt{9m^2 - 8l^4n}$$

$$R_O = \frac{wa}{2l^3}(m) \qquad R_B = \frac{wa^3}{2l^3}(2l - a)$$

$$M_O = -\frac{wa^2}{12l^2}(n) \qquad M_B = -\frac{wa^3}{12l^2}(4l - 3a)$$

$$M_A = \frac{wa^3}{12l^3}(6a^2 - 15al + 8l^2)$$

Bending moment:

(i) $x \leqq a$

$$M_x = \frac{w}{12l^3}\left[-6l^3x^2 + 6ax(m) - a^2l(n)\right]$$

(ii) $x \geqq a$

$$M_x = \frac{wa^3}{12l^3}\left[6x(a - 2l) - 3al + 8l^2\right]$$

$$M_{D'} = M_{\text{max. pos.}} = \frac{wa^2}{24l^6}\left(3m^2 - 2l^4n\right)$$

$$\text{at point where } x = OD = \frac{R_O}{w}$$

Equations to elastic line:

(i) $x \leqq a$

$$y = \frac{wx^2}{24EIl^3}\left[l^3x^2 - 2ax(m) + a^2l(n)\right]$$

(ii) $x \geqq a$

$$y = \frac{wa^3}{24EIl^3}\left[x^3(4l - 2a) - lx^2(8l - 3a) + l^3(4x - a)\right]$$

$$\text{Deflection at A} = \frac{wa^4}{24EIl^3}\left[a^2(7l - 2a) + l^2(3l - 8a)\right]$$

Maximum deflection: when it occurs for

(i) $x \leqq a$

$$\delta = \frac{wa^4(3m - p)^2}{6144\,EI\,l^{12}}\left[(3m - p)^2 - 8m(3m - p) + 16l^4n\right]$$

$$\text{at point where } x = \frac{a(3m - p)}{4l^3}$$

(ii) $x \geqq a$

$$\delta = \frac{wa^3}{648\,EI\,(2l - a)^2}\left[64l^3 - 144al^2 + 27a^2(4l - a)\right]$$

$$\text{at point where } x = \frac{2l^2}{3(2l - a)}$$

$\delta_{\text{max.}}$ occurs for $x = a$ when $a = .423\,l$

Points of inflection:

$$OF = \frac{a}{2l^3}\left[m - \sqrt{\tfrac{1}{3}(3m^2 - 2l^4n)}\right] \qquad OG = \frac{l(3a - 8l)}{6(a - 2l)}.$$

l = Span. W = Total Load. w = Load per Foot.
M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.
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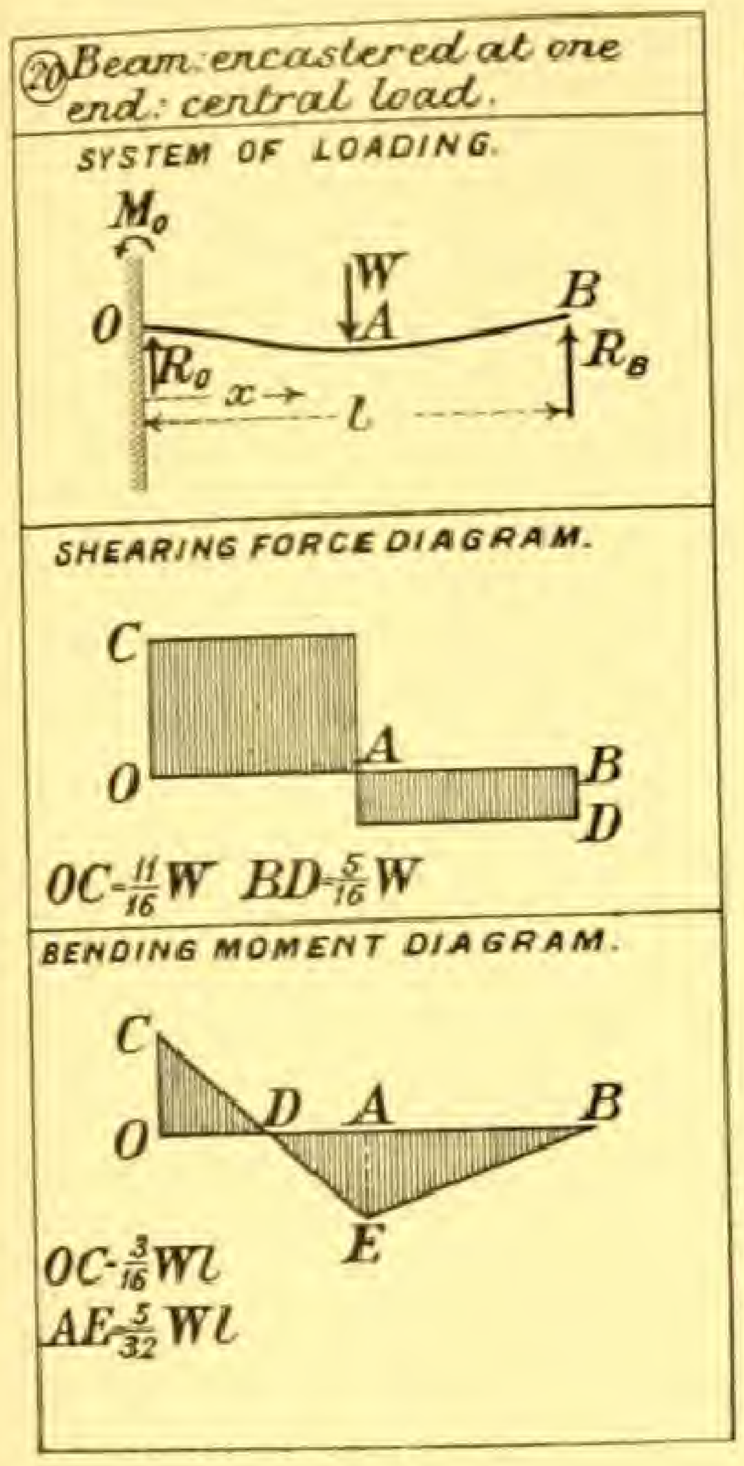
Expressions involving I apply only to beams having a constant Moment of Inertia.



(20) $R_O = \frac{11}{16} W$. $R_B = \frac{5}{16} W$

Bending moment:

(i) $x \leqq \frac{l}{2}$; $M_x = \frac{W}{16}\left(11x - 3l\right)$

(ii) $x \geqq \frac{l}{2}$; $M_x = \frac{5W}{16}\left(l - x\right)$

$M_O = -\frac{3}{16} Wl$. $M_B = 0$

$M_A = +\frac{5}{32} Wl$

Equations to elastic line:

(i) $x \leqq \frac{l}{2}$

$$y = \frac{Wx^2}{96\,EI}\left(9l - 11x\right)$$

(ii) $x \geqq \frac{l}{2}$

$$y = \frac{W}{96\,EI}\left(5x^3 - 15lx^2 + 12l^2x - 2l^3\right)$$

Deflection at centre $= \frac{7}{768}\frac{Wl^3}{EI}$

Maximum deflection $= 0.0093 \frac{Wl^3}{EI}$,

where $x = 0.553\,l$

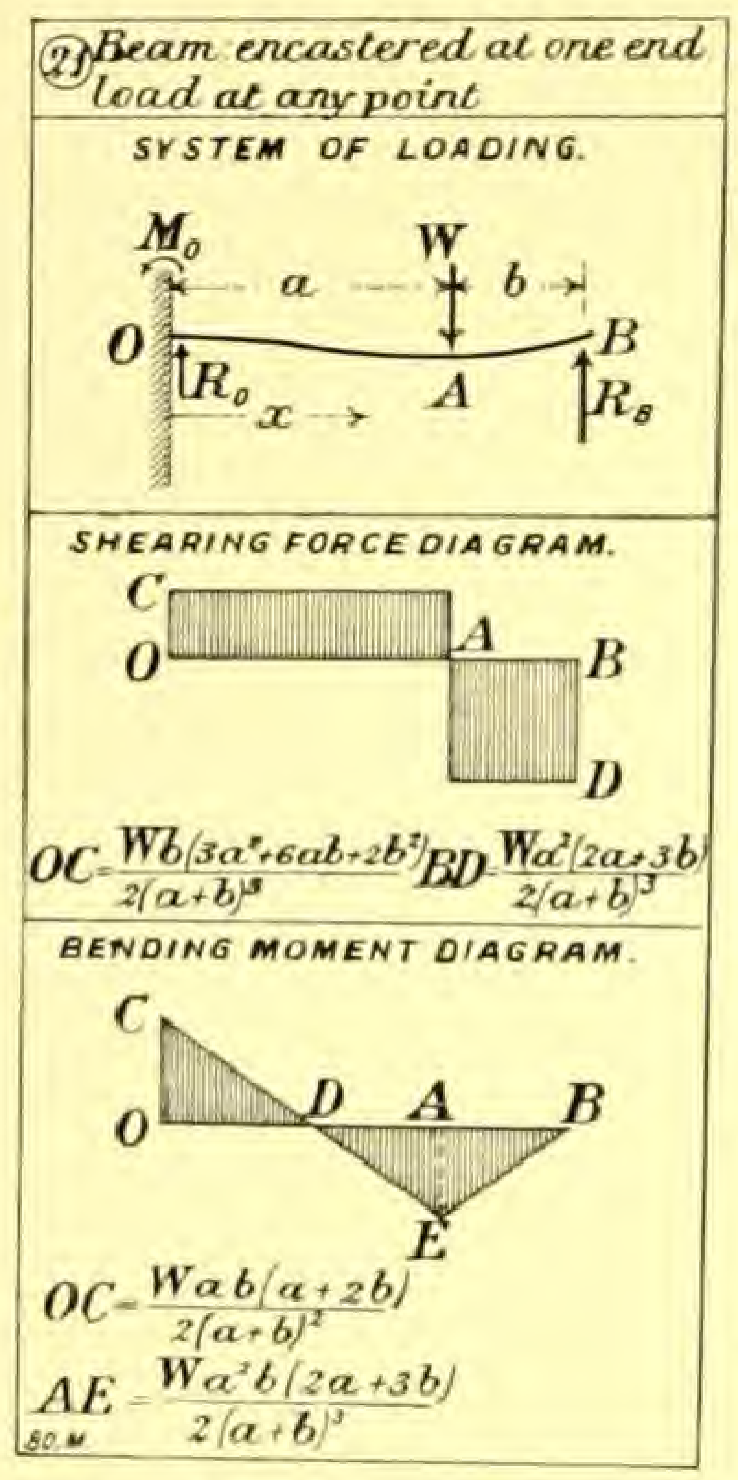
Point of inflection: $OD = \frac{3}{11} l$.

l = Span. W = Total Load. w = Load per Foot.

M_0, M_A, M_B Bending Moments at O, A, B. R_0, R_A, R_B Reactions at O, A, B.

E = Modulus of Elasticity. I = Moment of Inertia.

Expressions involving I apply only to beams having a constant Moment of Inertia.



(21)

$$R_0 = \frac{Wb(3a^2 + 6ab + 2b^2)}{2(a+b)^3}$$

$$R_B = \frac{Wa^2(2a + 3b)}{2(a+b)^3}$$

$$M_0 = -\frac{Wab(a + 2b)}{2(a+b)^2}$$

$$M_A = \frac{Wa^2b(2a + 3b)}{2(a+b)^3}$$

Bending moment:

(i) $x \leqq a$

$$M_x = \frac{Wb}{2(a+b)^3}\left[x(3a^2 + 6ab + 2b^2) - a(a^2 + 3ab + 2b^2)\right]$$

(ii) $x \geqq a$

$$M_x = \frac{W}{2(a+b)^3}\left[x(b^3 + 3ab^2 - a^3) - a(b^3 - 2a^2b - a^3)\right]$$

Equations to elastic line:

(i) $x \leqq a$

$$y = \frac{Wbx^2}{12(a+b)^3 EI}\left[-x(3a^2 + 6ab + 2b^2) + 3a(a^2 + 3ab + 2b^2)\right]$$

(ii) $x \geqq a$

$$y = \frac{Wa^2}{12(a+b)^3 EI}\left[\left\{x^3 - 3x^2(a+b)\right\}\left\{2a + 3b\right\} + (6x - 2a)(a+b)^3\right]$$

Deflection at A $= \dfrac{Wa^3b^2}{12(a+b)^3 EI}\left(3a + 4b\right)$

Maximum deflection $= \dfrac{Wa^2b}{6EI}\sqrt{\dfrac{b}{2a + 3b}}$

at point where $x = (a+b)\left\{1 - \sqrt{\dfrac{b}{2a+3b}}\right\}$

$\delta_{max.}$ occurs at $x = a$ when $b = .707\,a$

Point of inflection: $OD = \dfrac{a(a^2 + 3ab + 2b^2)}{3a^2 + 6ab + 2b^2}$.

l = Span. W = Total Load. w = Load per Foot.

M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.

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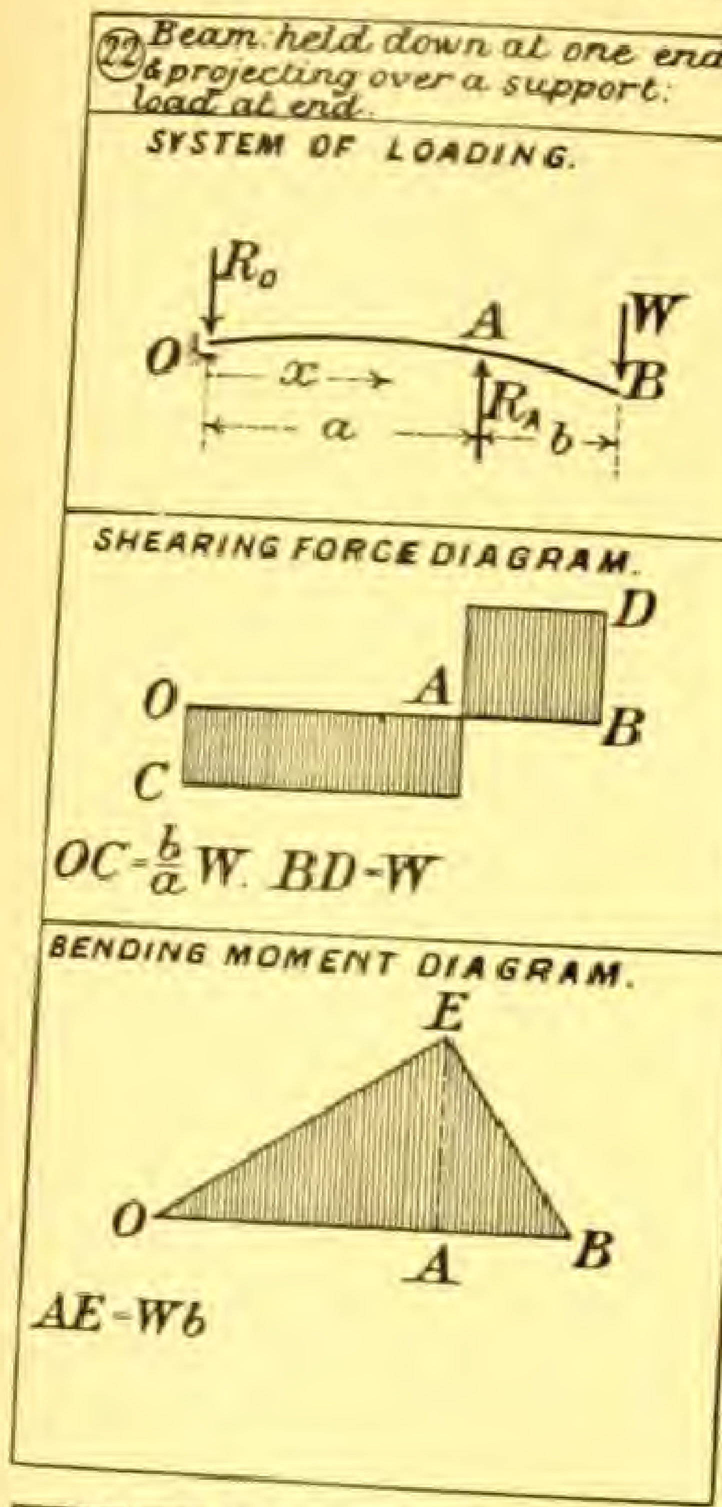


(22) $R_O = -\dfrac{Wb}{a}$. $R_A = \dfrac{W(a+b)}{a}$

Bending moment:

(i) $x \leqq a$. $M_x = -\dfrac{Wbx}{a}$

(ii) $x \geqq a$. $M_x = -W(a+b-x)$

$M_O = M_B = O$. $M_A = -Wb$

Equations to elastic line:

(i) $x \leqq a$

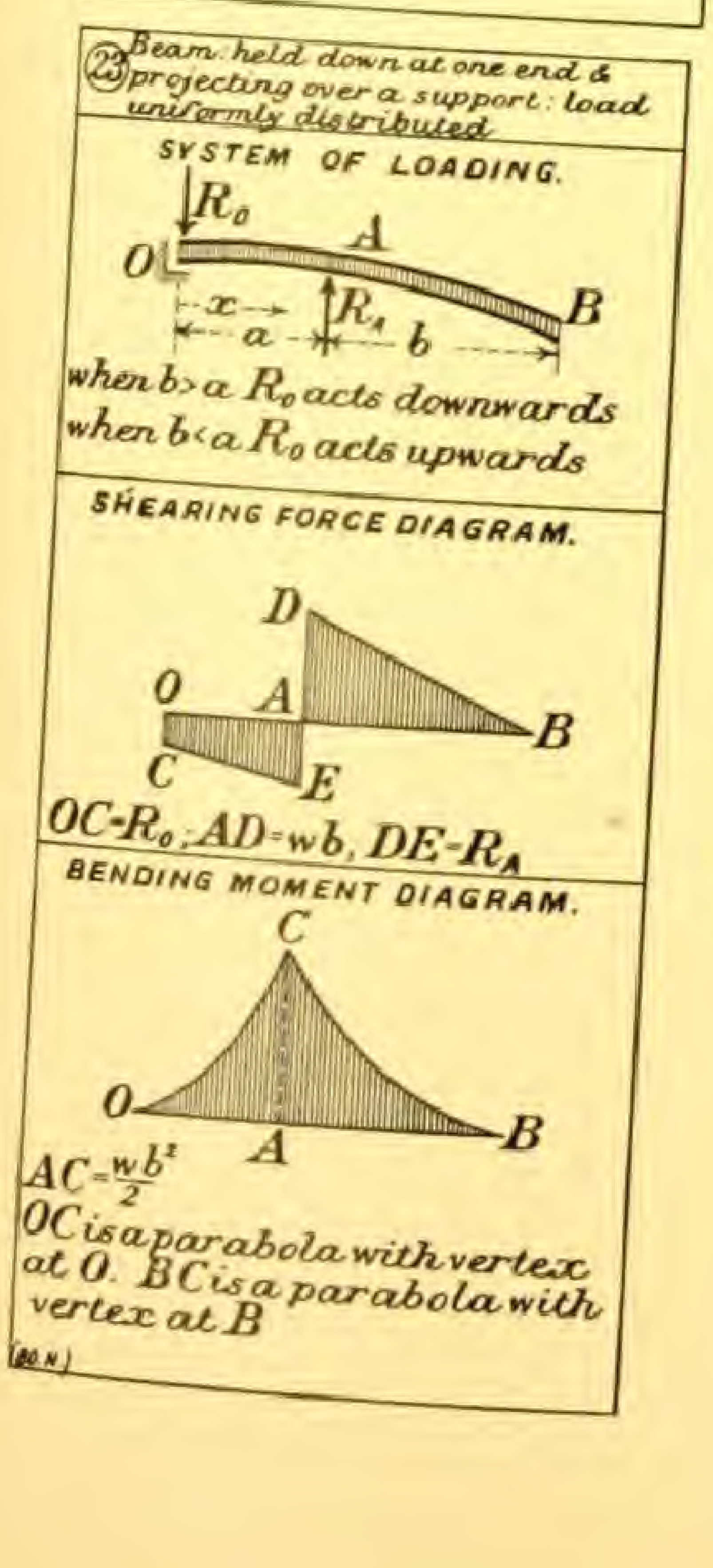
$$y = \frac{Wbx}{6\,EIa}\left(x^2 - a^2\right)$$

(ii) $x \geqq a$

$$y = \frac{W}{6\,EI}\left[(a-x)^3 + 3\,bx^2 - 4\,abx + a^2b\right]$$

$$\text{Deflection at B} = \frac{Wb^2}{3\,EI}\left(a+b\right)$$

Maximum negative deflection $= -0.0642\,\dfrac{Wa^2b}{EI}$; where $x = 0.577\,a$.



(23) $R_O = \dfrac{w}{2\,a}\left(a^2 - b^2\right)$. $R_A = \dfrac{w}{2\,a}\left(a+b\right)^2$

Bending moment:

(i) $x \leqq a$ $M_x = -\dfrac{w\,x}{2\,a}\left(b^2 + a\,x - a^2\right)$

(ii) $x \geqq a$ $M_x = -\dfrac{w}{2}\left(a + b - x\right)^2$

$M_A = -\dfrac{wb^2}{2}$

Equations to elastic line:

(i) $x \leqq a$

$$y = \frac{w}{24\,EIa}\left[ax^4 - 2\,x^3\,(a^2 - b^2) - a^2x\,(2\,b^2 - a^2)\right]$$

(ii) $x \geqq a$

$$y = \frac{w}{24\,EI}\left[x^4 - 4\,x^3\,(a+b) + 2\,(3\,x^2 + a^2)\,(a+b)^2 - ax\,(n)\right]$$

where $n = (5\,a^2 + 12\,ab + 8\,b^2)$

$$\text{Deflection at B} = \frac{wb}{24\,EI}\left(3\,b^3 + 4\,ab^2 - a^3\right)$$

Maximum negative deflection: it occurs at x in

$$4\,ax^3 - 6\,x^2(a^2 - b^2) - a^2(2\,b^2 - a^2) = O$$

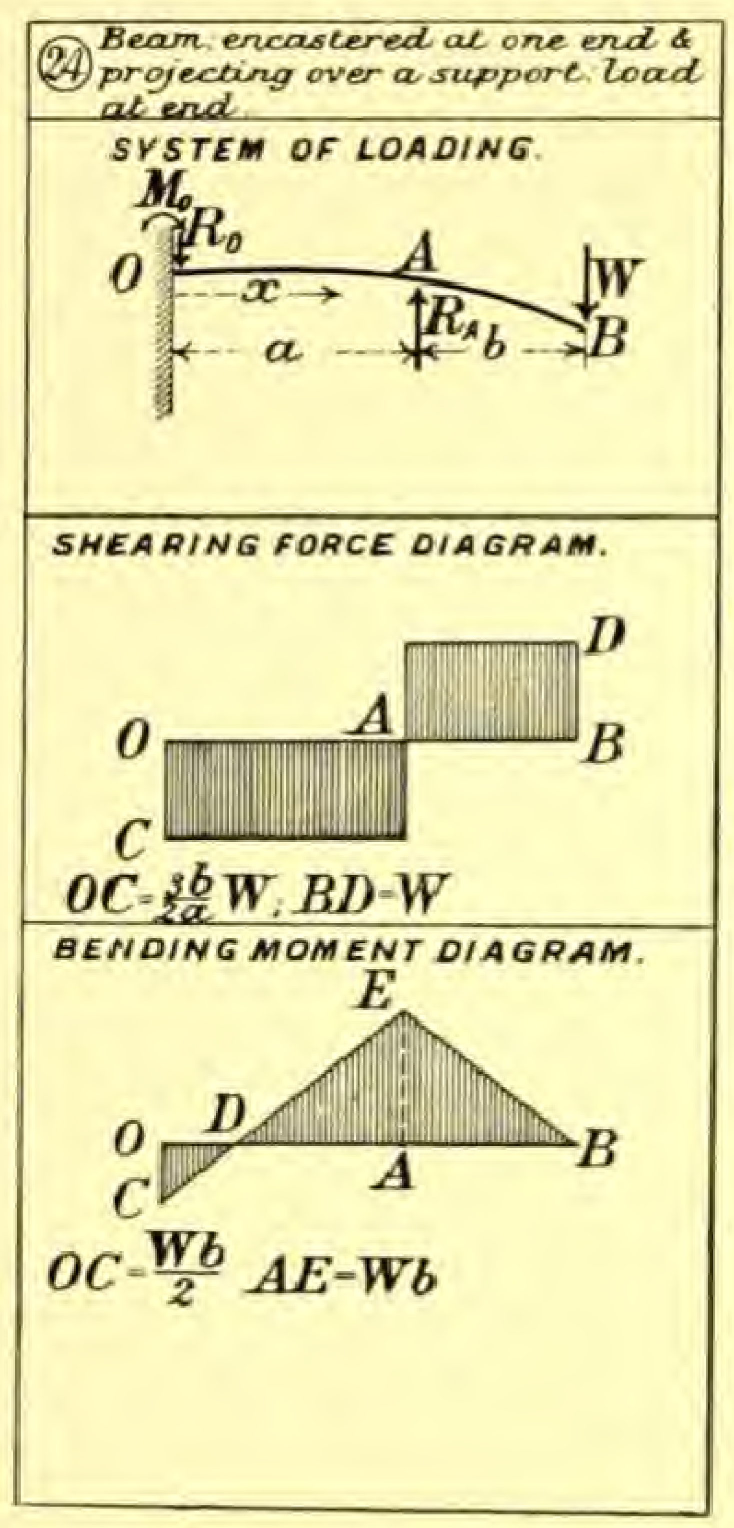
$\delta_{max.}$ is obtained by substituting this value of x in equation (i) for y.

l = Span. W = Total Load. w = Load per Foot.

M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.

E = Modulus of Elasticity. I = Moment of Inertia.

Expressions involving I apply only to beams having a constant Moment of Inertia.



(24) $R_A = W\left(1 + \dfrac{3b}{2a}\right)$. $R_O = -W\dfrac{3b}{2a}$

Bending moment:

(i) $x \leqq a$. $M_x = \dfrac{Wb}{2a}\left(a - 3x\right)$

(ii) $x \geqq a$. $M_x = W(x - a - b)$

$M_O = \dfrac{Wb}{2}$. $M_A = -Wb$. $M_B = 0$

Equations to elastic line:

(i) $x \leqq a$

$$y = \frac{Wbx^2}{4\,EIa}\left(x - a\right)$$

(ii) $x \geqq a$

$$y = \frac{W}{12\,EI}\left[2(a - x)^3 + 6bx^2 - 9abx + 3a^2b\right]$$

Deflection at B $= \dfrac{Wb^2}{12\,EI}\left(3a + 4b\right)$

Maximum negative deflection: $= -\dfrac{Wa^3b}{27\,EI}$

where $x = \dfrac{2}{3}a$

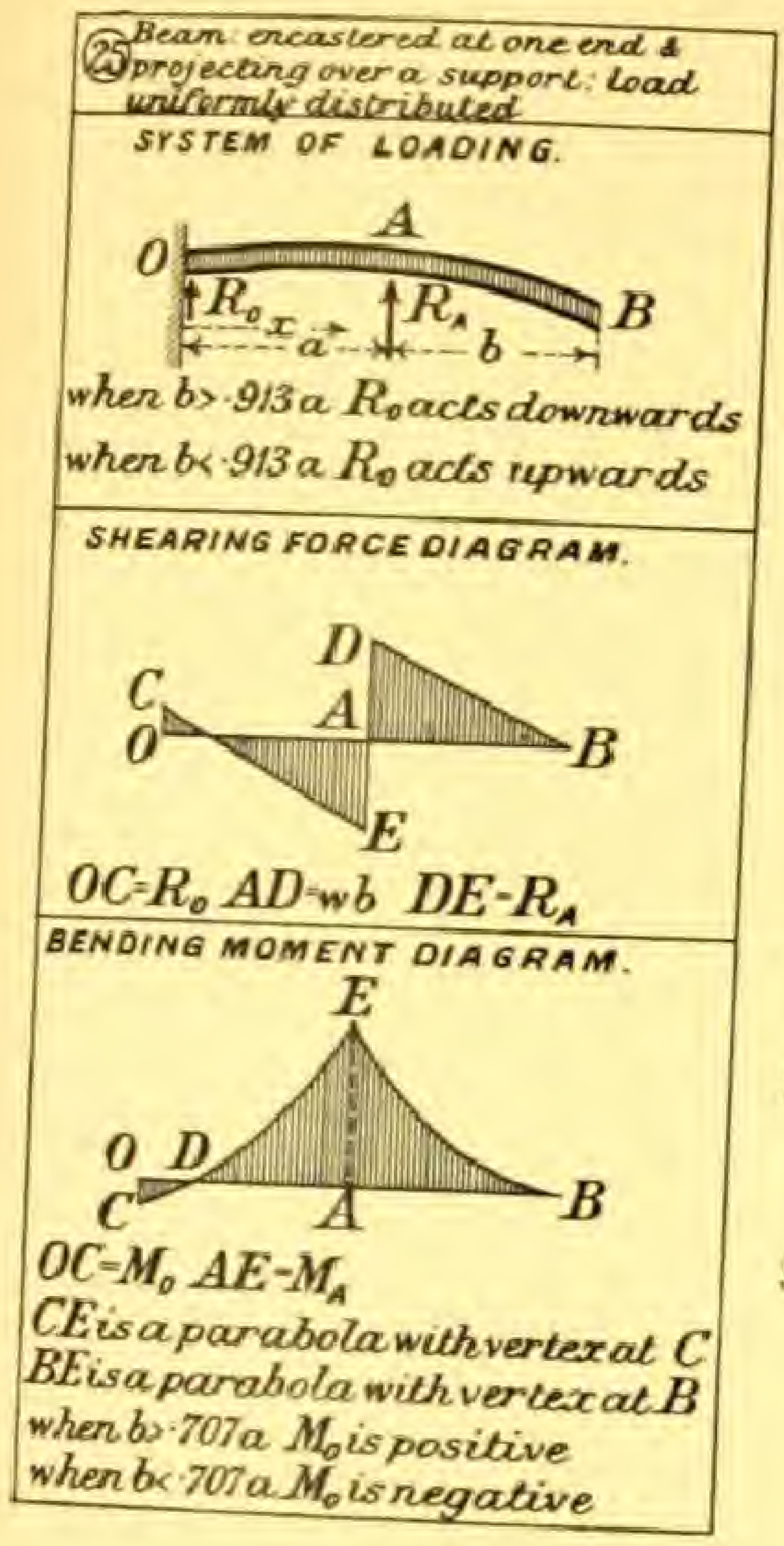
Point of inflection: $OD = \dfrac{a}{3}$.

l = Span. W = Total Load. w = Load per Foot.

M_o, M_A, M_B Bending Moments at O, A, B. R_o, R_A, R_B Reactions at O, A, B.

E = Modulus of Elasticity. I = Moment of Inertia.

Expressions involving I apply only to beams having a constant Moment of Inertia.



(25)

$$R_A = \frac{w}{8a}\left(3a^2 + 8ab + 6b^2\right)$$

$$R_o = \frac{w}{8a}\left(5a^2 - 6b^2\right)$$

Bending moment:

(i) $x \leqq a$

$$M_x = -\frac{w}{8a}\left[4ax^2 + (6b^2 - 5a^2)x - 2ab^2 + a^3\right]$$

(ii) $x \geqq a$

$$M_x = -\frac{w}{2}\left(a + b - x\right)^2$$

$$M_o = \frac{w}{8}\left(2b^2 - a^2\right) \quad . \quad M_A = -\frac{1}{2}wb^2$$

Equations to elastic line:

(i) $x \leqq a$

$$y = \frac{wx^2}{48\,\mathrm{EI}a}\left[2ax^2 + (6b^2 - 5a^2)x - 6ab^2 + 3a^3\right]$$

(ii) $x \geqq a$

$$y = \frac{w}{48\,\mathrm{EI}}\left[2x^4 - 8x^3(a + b) + 12x^2(a + b)^2 - 3ax(n) + a^2(n)\right]$$

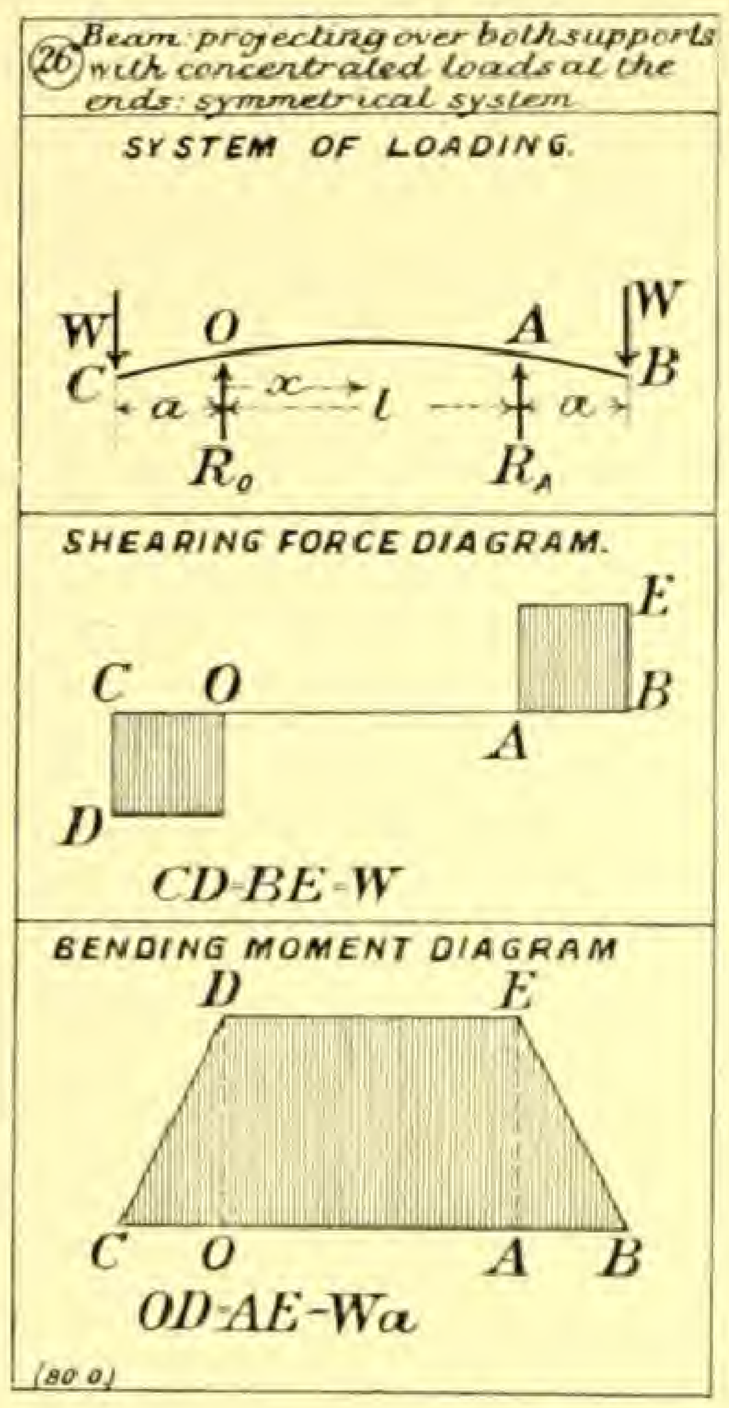
where $n = (3a^2 + 8ab + 6b^2)$

$$\text{Deflection at B} = \frac{wb}{48\,\mathrm{EI}}\left(6b^3 + 6ab^2 - a^3\right)$$

Point of inflection:

$$\mathrm{OD} = \frac{5a^2 - 6b^2 + \sqrt{9a^4 - 28a^2b^2 + 36b^4}}{8a}$$

l = Span. W = Total Load. w = Load per Foot.
M_0, M_A, M_B Bending Moments at O, A, B. R_0, R_A, R_B Reactions at O, A, B.
E = Modulus of Elasticity. I = Moment of Inertia
Expressions involving I apply only to beams having a constant Moment of Inertia.



(26)

$$R_0 = R_A = W$$

Shearing force between O and C = $-W$
Shearing force between O and A = zero

$$M_0 = M_A = -Wa$$

Equations to elastic line:

(i) $x \begin{matrix} > o \\ < l \end{matrix}$

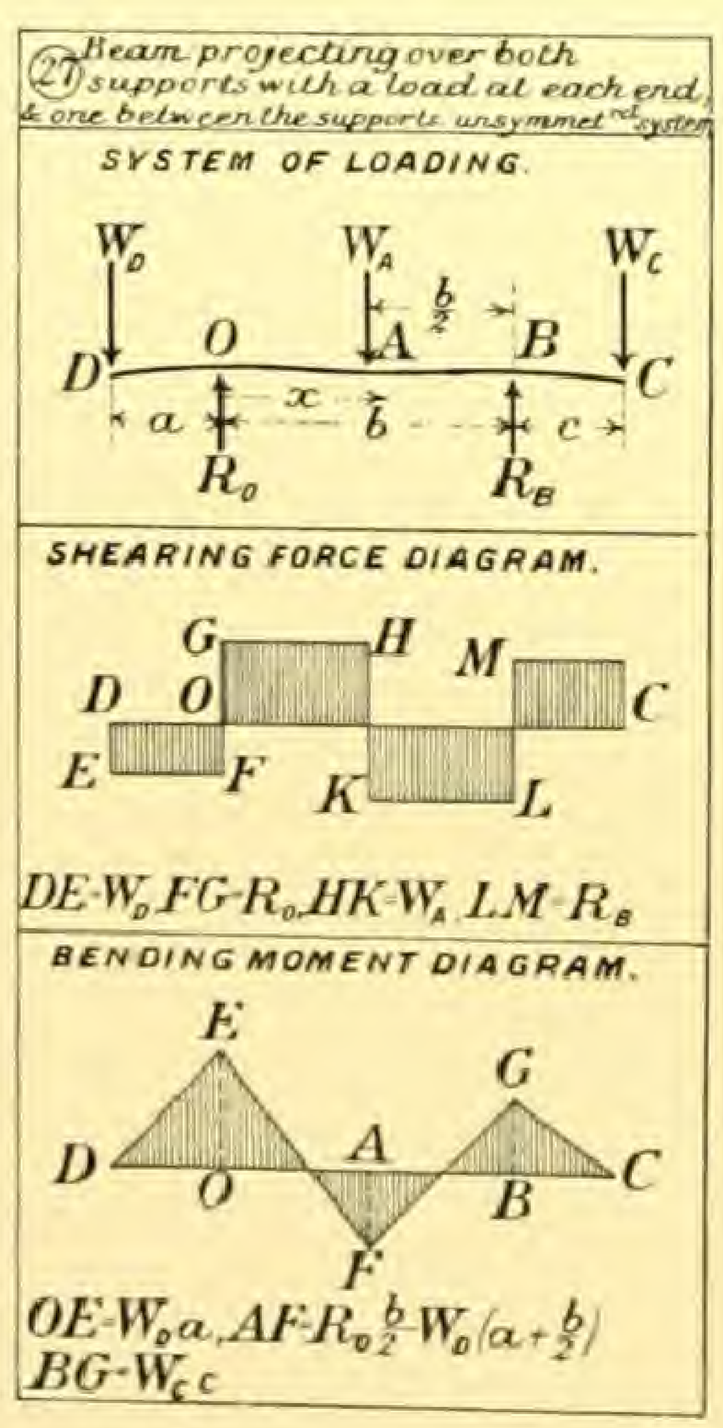
$$y = -\frac{Wax}{2\,EI}\left(l - x\right)$$

(ii) $x \begin{matrix} < o \\ > -a \end{matrix}$

$$y = \frac{Wx}{6\,EI}\left(x^2 + 3\,ax - 3\,al\right)$$

Deflection at centre $= -\dfrac{Wal^2}{8\,EI}$

Deflection at C or B $= \dfrac{Wa^2}{6\,EI}\left(2\,a + 3\,l\right)$.



(27)

$$R_0 = \frac{1}{b}\left\{W_D\,(a + b) + W_A\frac{b}{2} - W_C\,c\right\}$$

$$R_B = \frac{1}{b}\left\{W_C\,(b + c) + W_A\frac{b}{2} - W_D\,a\right\}$$

Bending moment:

(i) $x \leqq o$ and $\geqq (-a)$. $M_x = -W_D\,(a + x)$

(ii) $x \geqq o$ and $\leqq \dfrac{b}{2}$. $M_x = R_0\,x - W_D\,(a + x)$

$M_0 = -W_D\,a$. $M_B = -W_C\,c$

$$M_A = \frac{1}{4}\left(W_A\,b - 2\,W_C\,c - 2\,W_D\,a\right)$$

Equations to elastic line:

(i) $x \leqq o$ and $\geqq (-a)$

$$y = \frac{1}{48\,EI}\left\{8\,W_D\,x^3 + 24\,W_D\,ax^2 + (m)\,bx\right\}$$

(ii) $x \geqq o$ and $\leqq \dfrac{b}{2}$

$$y = \frac{1}{48\,EI}\left\{(-W_A\,b + 2\,W_C\,c - 2\,W_D\,a)\frac{4\,x^3}{b} + 24\,W_D\,ax^2 + (m)\,bx\right\}$$

where $m = (3\,W_A\,b - 8\,W_C\,c - 16\,W_D\,a)$

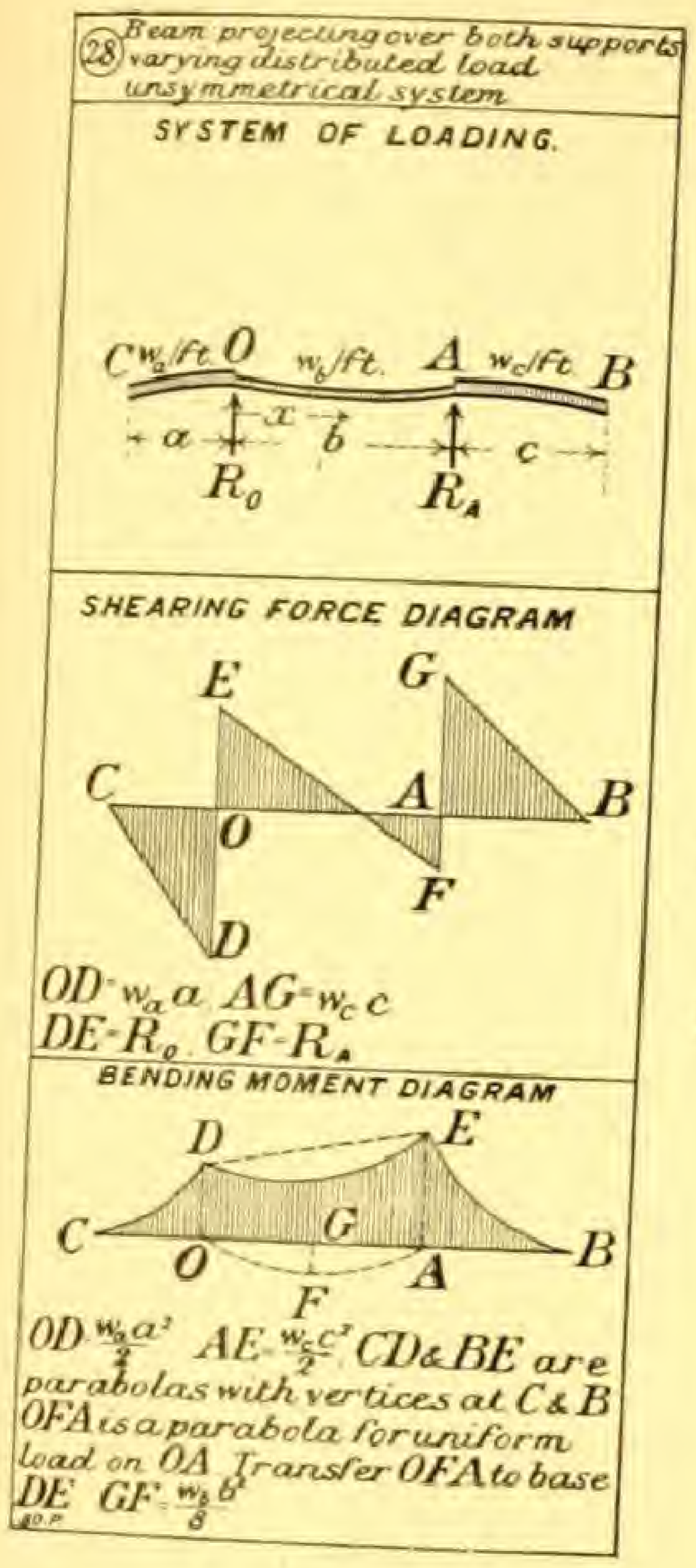
Deflections at A, C and D:

$$\delta_A = \frac{b^2}{48\,EI}\left(W_A\,b - 3\,W_C\,c - 3\,W_D\,a\right)$$

$$\delta_C = \frac{c}{48\,EI}\left\{16\,W_C\,c\,(b + c) + 8\,W_D\,ab - 3\,W_A\,b^2\right\}$$

$$\delta_D = \frac{a}{48\,EI}\left\{16\,W_D\,a\,(a + b) + 8\,W_C\,bc - 3\,W_A\,b^2\right\}.$$

l = Span. W = Total Load. w = Load per Foot.
M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.
E = Modulus of Elasticity. I = Moment of Inertia.
Expressions involving I apply only to beams having a constant Moment of Inertia.



(28)

$$R_O = \frac{1}{2b}\left\{w_a a\,(a + 2b) + w_b b^2 - w_c c^2\right\}$$

$$R_A = \frac{1}{2b}\left\{w_c c\,(c + 2b) + w_b b^2 - w_a a^2\right\}$$

Bending moment :

(i) $x \leqq o$ and $\geqq -a$. $M_x = -\frac{w_a}{2}\left(a + x\right)^2$

(ii) $x \geqq o$ and $\leqq b$. $M_x = R_O x - \frac{1}{2}\left(2 w_a a x + w_a a^2 - w_b x^2\right)$

$$M_O = -\frac{w_a a^2}{2} \qquad . \qquad M_A = -\frac{w_c c^2}{2}$$

Equations to elastic line :

(i) $x \leqq o$ and $\geqq -a$.

$$y = \frac{1}{24\,EI}\left\{w_a\,(x^4 + 4ax^3 + 6a^2x^2) + (m)\,x\right\}$$

(ii) $x \geqq o$ and $\leqq b$

$$y = \frac{1}{24\,EI}\left\{w_b x^4 - (w_a a^2 + w_b b^2 - w_c c^2)\frac{2x^3}{b} + 6 w_a a^2 x^2 + (m)\,x\right\}$$

where $m = (-4 w_a a^2 b + w_b b^3 - 2 w_c b c^2)$

Deflections at $\frac{b}{2}$, B and C :

$$\delta_{\frac{b}{2}} = \frac{b^2}{384\,EI}\left(5 w_b b^2 - 12 w_a a^2 - 12 w_c c^2\right)$$

$$\delta_B = \frac{c}{24\,EI}\left\{w_c c^2 (3c + 4b) - w_b b^3 + 2 w_a a^2 b\right\}$$

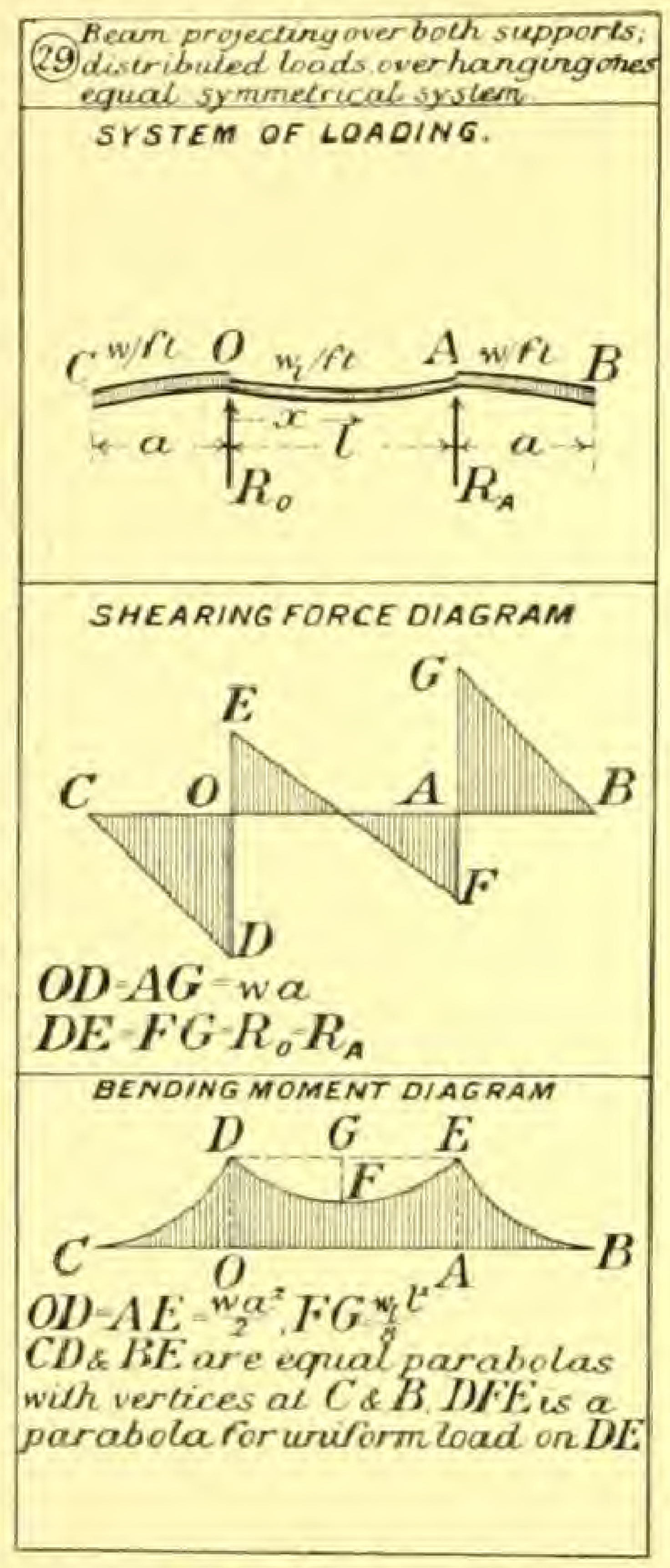
$$\delta_C = \frac{a}{24\,EI}\left\{w_a a^2 (3a + 4b) - w_b b^3 + 2 w_c b c^2\right\}.$$

l = Span. W = Total Load. w = Load per Foot.

M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.

E = Modulus of Elasticity. I = Moment of Inertia.

Expressions involving I apply only to beams having a constant Moment of Inertia.



(29) $$R_O = R_A = \frac{1}{2}\left(2\,wa + w_1 l\right)$$

Bending moment:

(i) $x \leqq o$ and $\geqq -a$. $$M_x = -\frac{w}{2}\left(a + x\right)^2$$

(ii) $x \geqq o$ and $\leqq l$. $$M_x = \frac{1}{2}\left(w_1 lx - w_1 x^2 - wa^2\right)$$

$$M_O = M_A = -\frac{wa^2}{2}$$

Equations to elastic line:

(i) $x \leqq o$ and $\geqq -a$

$$y = \frac{1}{24\,\mathrm{EI}}\left\{w\left(x^4 + 4\,ax^3 + 6\,a^2x^2\right) + \left(w_1 l^3 - 6\,wa^2 l\right)x\right\}$$

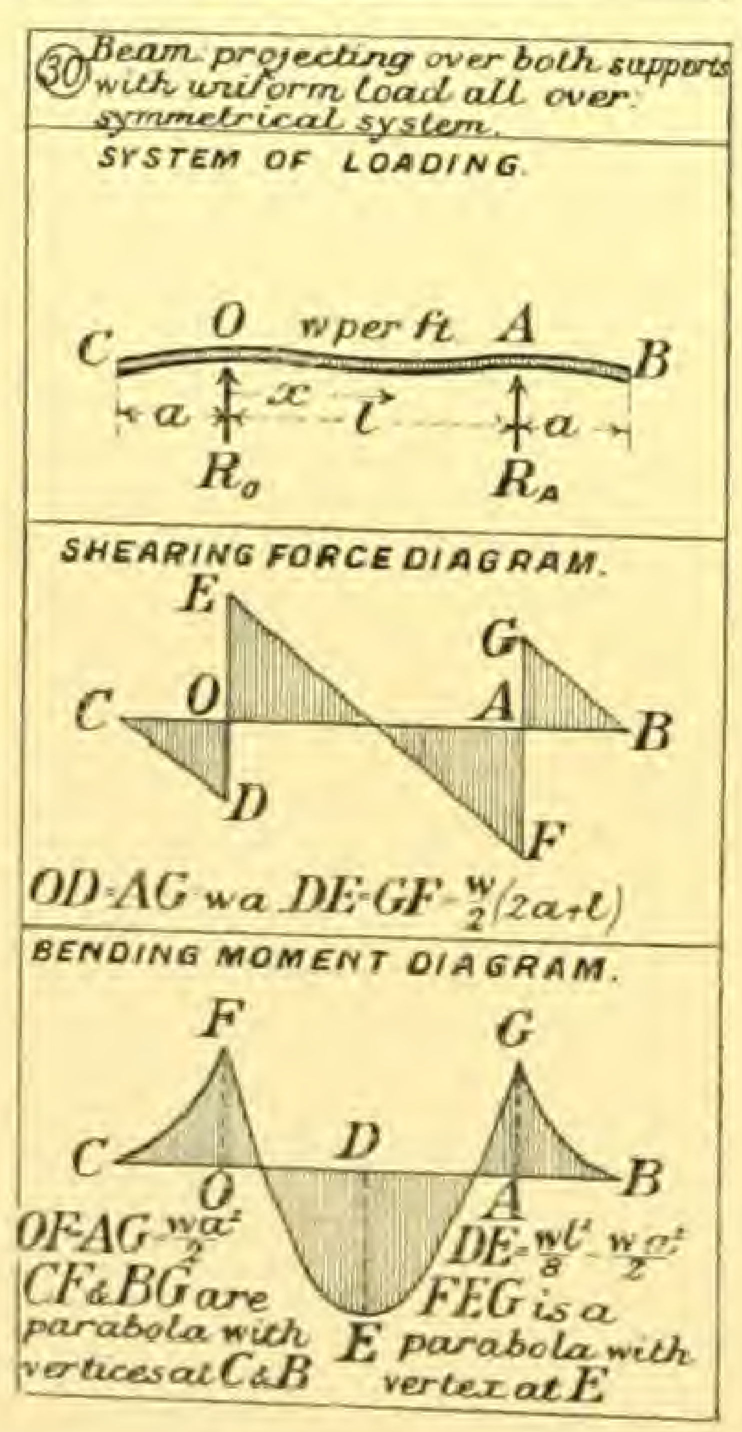
(ii) $x \geqq o$ and $\leqq l$

$$y = \frac{1}{24\,\mathrm{EI}}\left\{w_1\left(x^4 - 2\,lx^3\right) + 6\,wa^2x^2 + \left(w_1 l^3 - 6\,wa^2 l\right)x\right\}$$

Deflections at $\frac{l}{2}$, B and C:

$$\delta_{\frac{l}{2}} = \frac{l^2}{384\,\mathrm{EI}}\left(5\,w_1 l^2 - 24\,wa^2\right)$$

$$\delta_B = \delta_C = \frac{a}{24\,\mathrm{EI}}\left\{3\,w\left(a^3 + 2\,a^2 l\right) - w_1 l^3\right\}$$



(30) $$R_O = R_A = \frac{w}{2}\left(2\,a + l\right)$$

Bending moment:

(i) $x \leqq o$ and $\geqq -a$. $$M_x = -\frac{w}{2}\left(a + x\right)^2$$

(ii) $x \geqq o$ and $\leqq l$. $$M_x = -\frac{w}{2}\left(x^2 - lx + a^2\right)$$

$$M_O = M_A = -\frac{wa^2}{2} \quad . \quad M_D = \frac{w}{8}\left(l^2 - 4\,a^2\right)$$

Equations to elastic line:

(i) $x \leqq o$ and $\geqq -a$

$$y = \frac{w}{24\,\mathrm{EI}}\left\{x^4 + 4\,ax^3 + 6\,a^2x^2 + \left(l^3 - 6\,a^2 l\right)x\right\}$$

(ii) $x \geqq o$ and $\leqq l$

$$y = \frac{w}{24\,\mathrm{EI}}\left\{x^4 - 2\,lx^3 + 6\,a^2x^2 + \left(l^3 - 6\,a^2 l\right)x\right\}$$

Deflections at $\frac{l}{2}$, B and C:

$$\delta_{\frac{l}{2}} = \frac{wl^2}{384\,\mathrm{EI}}\left(5\,l^2 - 24\,a^2\right)$$

$$\delta_B = \delta_C = \frac{wa}{24\,\mathrm{EI}}\left(3\,a^3 + 6\,a^2 l - l^3\right).$$

Continuous Girders.

CONTINUOUS girders are not now generally employed except for swing bridges. This is chiefly owing to the effect that any change in the level of the supports of the girder may have on the calculated stresses. It is not necessary that the supports should be on a horizontal line, but they must be at such levels as will ensure the reactions being as required by theory. In practice this can be obtained by actually weighing the girders at each of their bearings and adjusting the levels until the correct reactions result. This method was adopted for the viaduct approach spans to the Forth Bridge. In calculating the stresses in continuous girders it is usual to assume that the moment of inertia of the girder is constant. This assumption is rarely correct, but the error caused by it is not of great importance, especially as the ordinary formulæ give somewhat higher stresses than those that would be obtained had a variable moment of inertia been considered. The effect of a difference of temperature on the various members of a girder may affect the stresses considerably. This does not admit of theoretical investigation of much value, but it indicates that great refinement in calculations of this kind are unnecessary. It would appear from the investigations of M. Lévy[1] that a difference in temperature of 25 deg. Fahr. between the flanges of a continuous girder may increase the stresses by nearly 2 tons per square inch, and this, in his opinion, makes it preferable to use independent spans in place of continuous ones, where appreciable differences of temperature between the upper and lower members of girders can occur.

It should also be remembered that riveted joints, defective detail, and many other matters may cause the actual stresses in a girder of any type to differ very considerably from those obtained by calculation, as shown by experiments on a large bridge over the Loire at Cosne.[2]

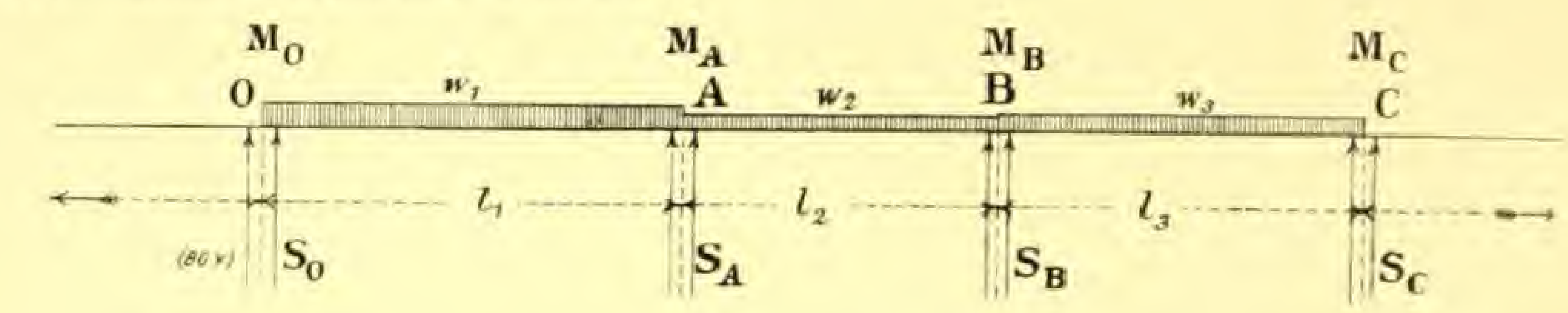
The bending moments and reactions of a continuous girder, of any number of spans, and loaded with concentrated or distributed loads, can be obtained by the theorem of three moments, which is as follows:—

1 "Le Génie Civil," September 28th, 1895.
2 "Annales des Ponts et Chaussées," November, 1895.

THE THREE-MOMENT THEOREM.

Let l_1 l_2 l_3 etc. = consecutive spans of a continuous girder
w_1 w_2 w_3 = distributed loads per unit of length on these spans
W_1 W_2 W_3 = concentrated loads on these spans.
M_0 M_A M_B M_C = bending moments at supports O, A, B and C
S_0 S_A S_B S_C = shearing force at supports O, A, B, C
h_0 h_A h_B h_C = vertical movement of supports O, A, B, C.

CASE (*a*).—*Loads Uniformly Distributed.*

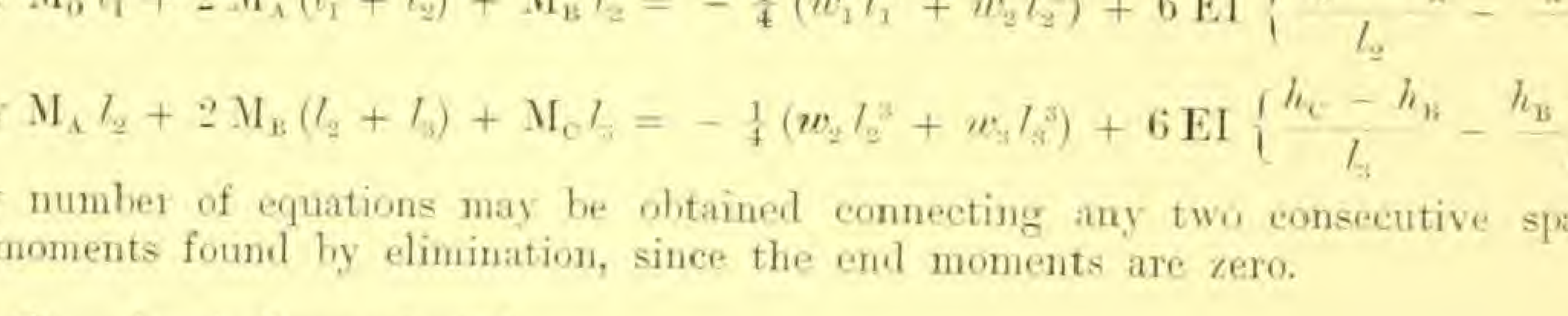


$$\text{Then } M_0 l_1 + 2 M_A (l_1 + l_2) + M_B l_2 = -\tfrac{1}{4}(w_1 l_1^3 + w_2 l_2^3) + 6\,EI\left\{\frac{h_B - h_A}{l_2} - \frac{h_A - h_0}{l_1}\right\}$$

$$\text{Similarly } M_A l_2 + 2 M_B (l_2 + l_3) + M_C l_3 = -\tfrac{1}{4}(w_2 l_2^3 + w_3 l_3^3) + 6\,EI\left\{\frac{h_C - h_B}{l_3} - \frac{h_B - h_A}{l_2}\right\}$$

Thus any number of equations may be obtained connecting any two consecutive spans, and the required moments found by elimination, since the end moments are zero.

CASE (*b*).—*Loads Concentrated.*

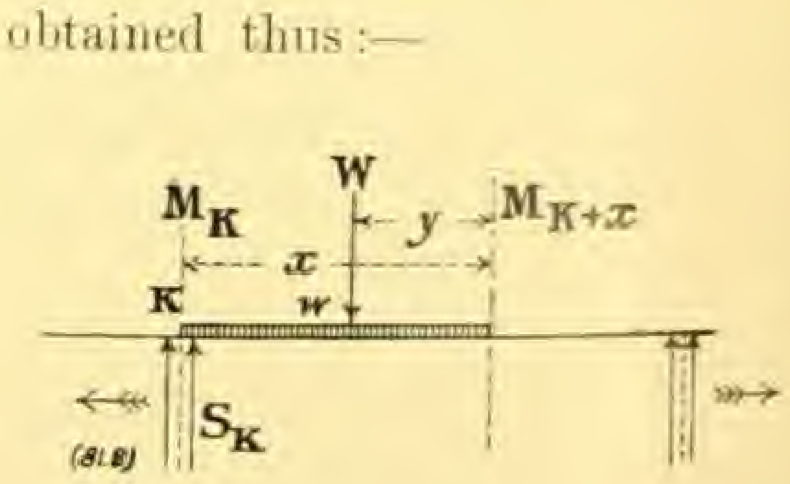


$$M_0 l_1 + 2 M_A (l_1 + l_2) + M_B l_2 = -\Sigma W_1 l_1^2 (n - n^3) - \Sigma W_2 l_2^2 (2n - 3n^2 + n^3) + + 6\,EI\left\{\frac{h_B - h_A}{l_2} - \frac{h_A - h_0}{l_1}\right\}$$

Similarly obtain any number of equations. See Note, Case (*a*).

The last term in all the above equations refers only to the effect of a vertical movement of the points of support, and can generally be rejected.[1]

Shearing Force.—The shearing force at any section may be obtained thus:—
Let M_K = bending moment at any section
S_K = shear at the section
M_{K+x} = bending moment at any other section
W and w = intervening external forces



$$\text{Then } M_{K+x} = M_K + S_K x - Wy - \frac{wx^2}{2}$$

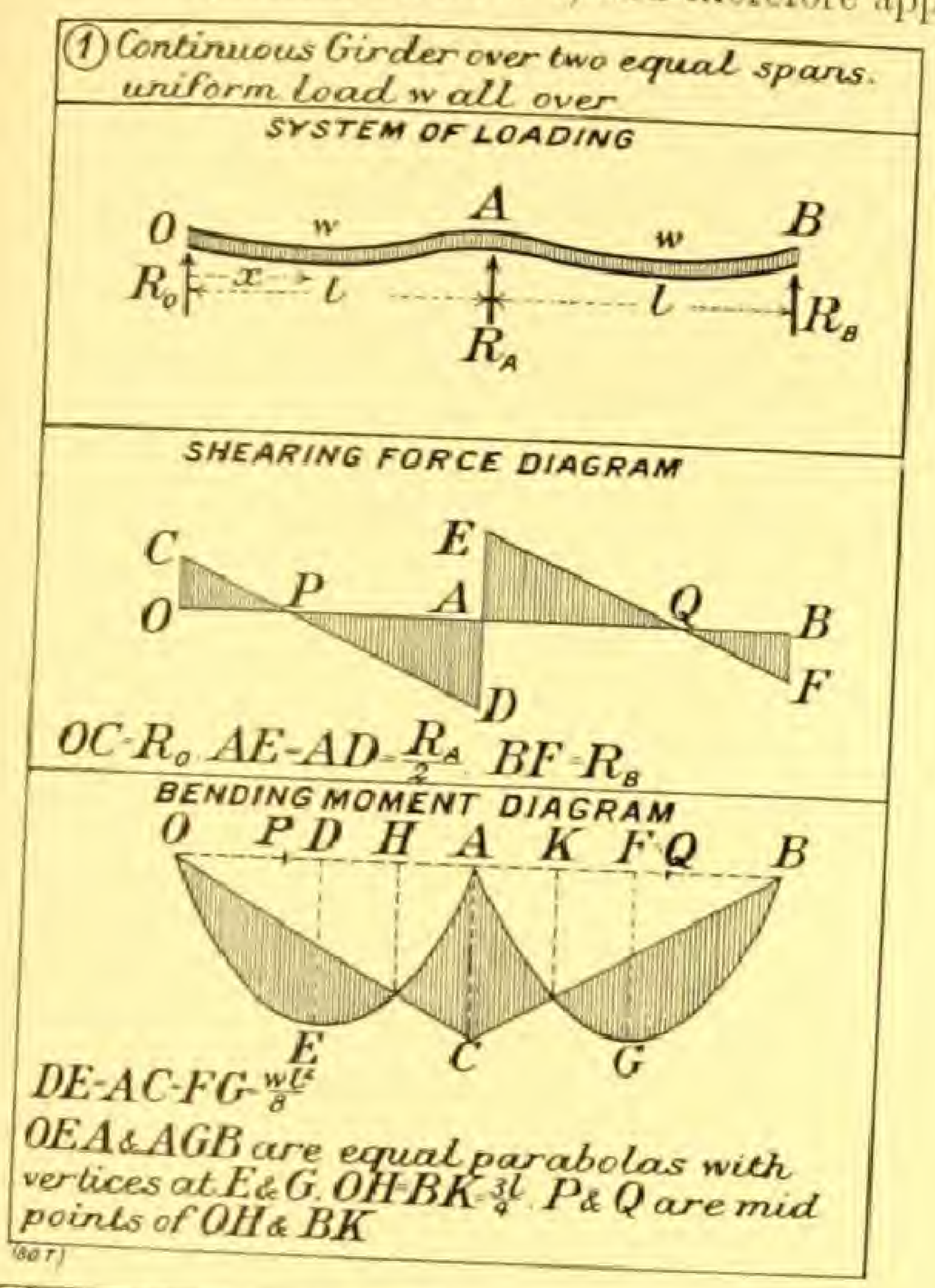
$$\text{That is } S_K = \frac{1}{x}\left(M_{K+x} - M_K + Wy + \frac{wx^2}{2}\right)$$

In the examples which follow the above equations have been applied, and it is assumed that no vertical movement occurs at the supports.

[1] For a complete mathematical investigation of the theorem of three moments, and its practical application, see "Modern Framed Structures," by Johnson, Bryan, and Turneaure (Wiley and Sons, New York).

l, l_1, l_2, l_3 = Spans. W, W_1, W_2, W_3 = Concentrated Loads. w, w_1, w_2, w_3 = Loads per Foot.
M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.
E = Modulus of Elasticity. I = Moment of Inertia.

All expressions involve I, and therefore apply only to beams having a constant Moment of Inertia.



(1)

$$R_O = R_B = \tfrac{3}{8}wl$$
$$R_A = \tfrac{5}{4}wl$$

Bending moment: $x \leq l$

$$M_x = \frac{w}{8}\left(3\,lx - 4\,x^2\right)$$
$$M_O = M_B = 0$$
$$M_A = -\frac{wl^2}{8}$$

Equation to elastic line: $x \leq l$

$$y = \frac{wx}{48\,EI}\left(l^3 - 3lx^2 + 2x^3\right)$$

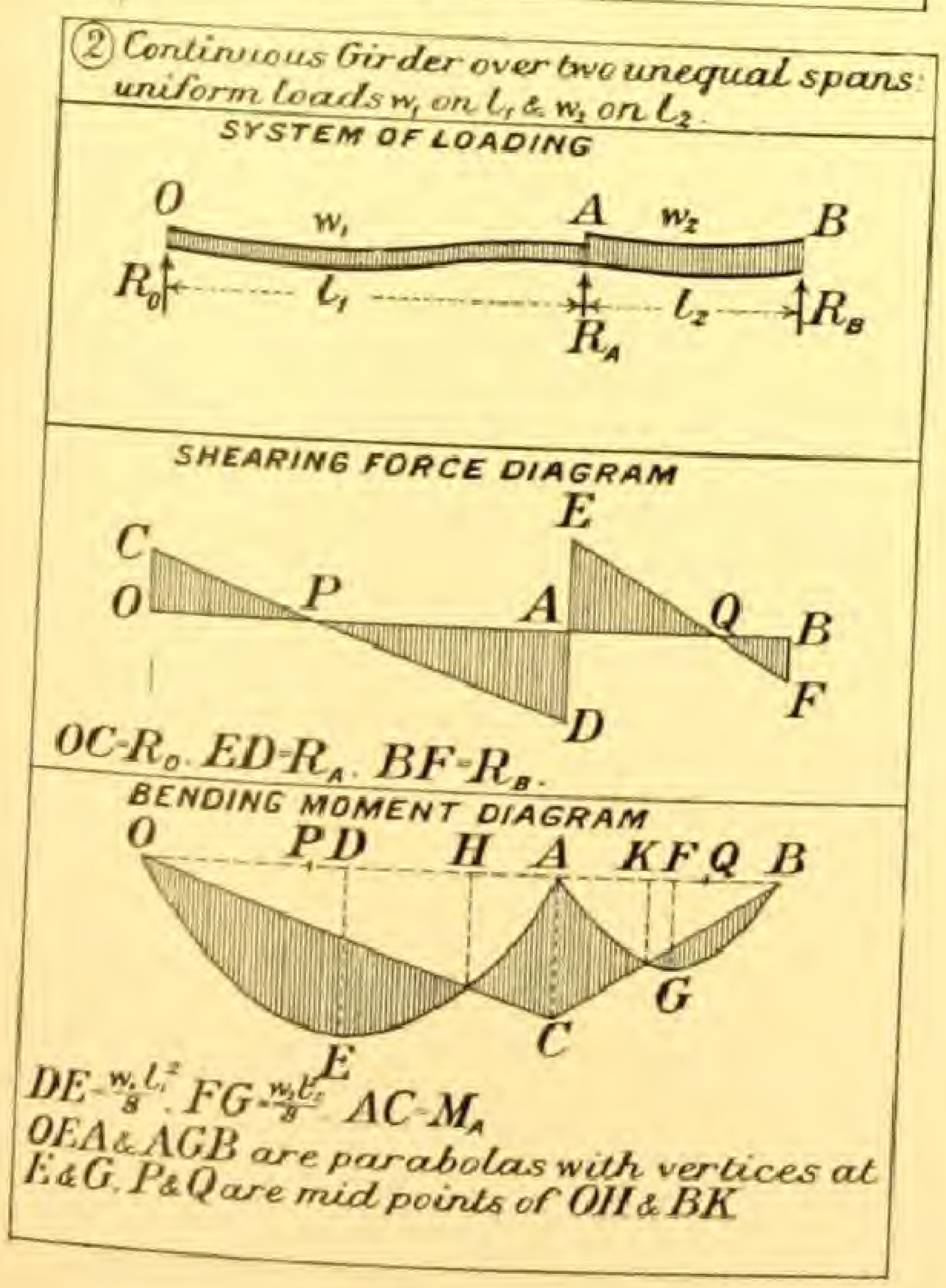
Deflection at centres of spans $= \dfrac{wl^4}{192\,EI}$

Maximum deflection $= 0.0054\,\dfrac{wl^4}{EI}$,

where $x = 0.421\,l$

Points of inflection occur at H and K, where

$$OH = BK = \frac{3\,l}{4}.$$



(2)

$$R_O = \frac{1}{l_1}\left(\frac{w_1 l_1^2}{2} - \frac{w_1 l_1^3 + w_2 l_2^3}{8\,(l_1 + l_2)}\right)$$
$$R_A = (w_1 l_1 + w_2 l_2) - (R_O + R_B)$$
$$R_B = \frac{1}{l_2}\left(\frac{w_2 l_2^2}{2} - \frac{w_1 l_1^3 + w_2 l_2^3}{8\,(l_1 + l_2)}\right)$$

Bending moment: $x \leq l_1$

$$M_x = R_O\,x - \frac{w_1 x^2}{2}$$
$$M_O = M_B = 0$$
$$M_A = -\frac{w_1 l_1^3 + w_2 l_2^3}{8\,(l_1 + l_2)}$$

Equation to elastic line: $x \leq l_1$

$$y = \frac{1}{24\,EI}\left\{w_1 x^4 - 4\,R_O x^3 + l_1^2 x\,(4\,R_O - w_1 l_1)\right\}$$

Maximum deflection on span l_1 is found from

$$4\,w_1 x^3 - 12\,R_O x^2 + 4\,R_O\,l_1^2 - w_1 l_1^3 = 0$$

Points of inflection occur at H and K, where

$$OH = \frac{2\,R_O}{w_1} \text{ and } BK = \frac{2\,R_B}{w_2}.$$

l, l_1, l_2, l_3 = Spans. W, W_1, W_2, W_3 = Concentrated Loads. w, w_1, w_2, w_3 = Loads per Foot.
M_0, M_A, M_B Bending Moments at O, A, B. R_0, R_A, R_B Reactions at O, A, B.
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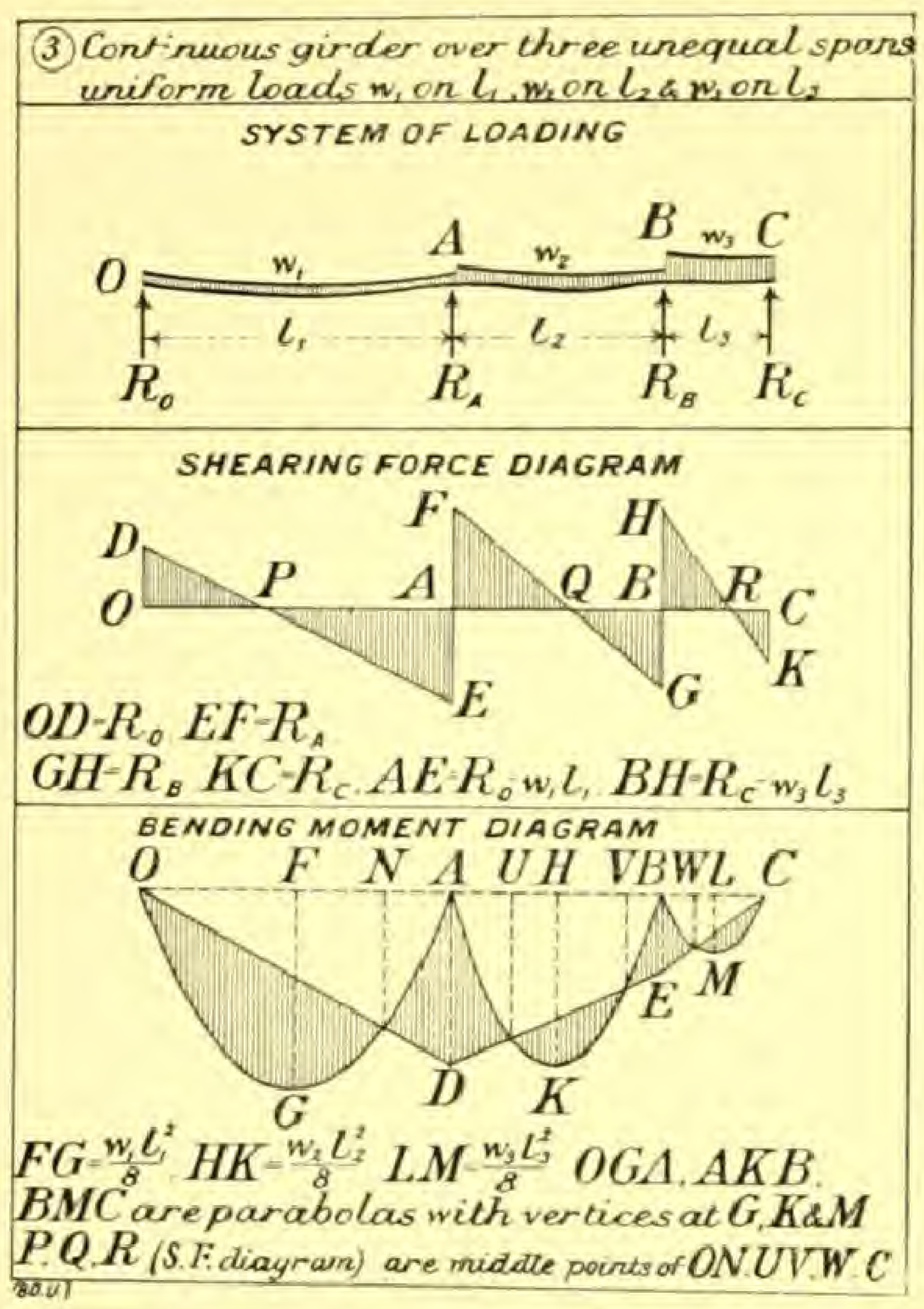
All expressions involve I, and therefore apply only to beams having a constant Moment of Inertia.



(3)

$$R_0 = \frac{1}{l_1}\left\{\frac{w_1 l_1^2}{2} + M_A\right\}$$

$$R_A = \frac{w_1 l_1}{2} + \frac{w_2 l_2}{2} - \frac{M_A}{l_1} - \frac{M_A - M_B}{l_2}$$

$$R_B = \frac{w_3 l_3}{2} + \frac{w_2 l_2}{2} - \frac{M_B}{l_3} - \frac{M_B - M_A}{l_2}$$

$$R_C = \frac{1}{l_3}\left\{\frac{w_3 l_3^2}{2} + M_B\right\}$$

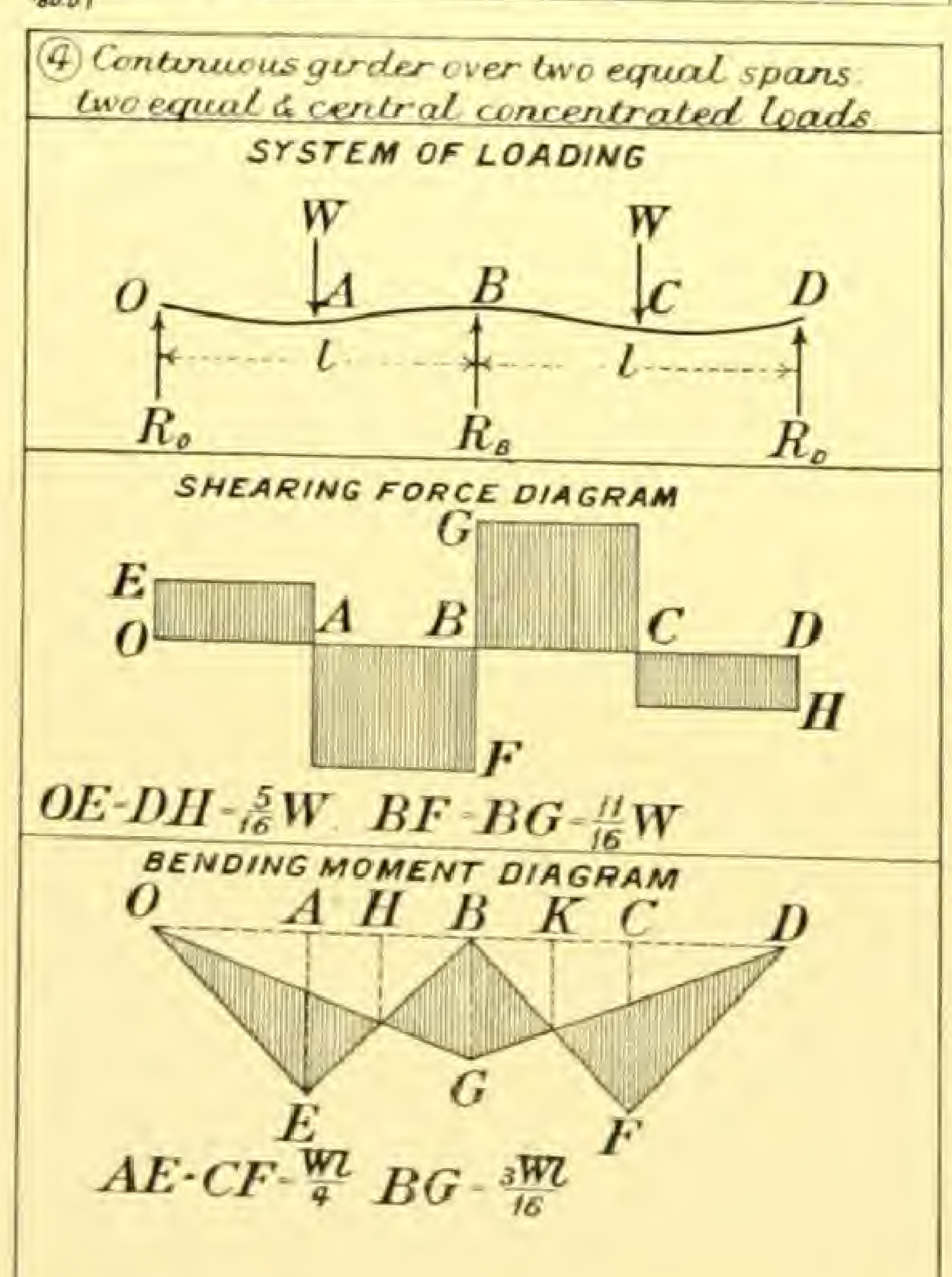
$$M_0 = M_C = O$$

$$M_A = -\frac{2 w_1 l_1^3 (l_2 + l_3) - w_2 l_2^3 (l_2 + 2 l_3) + w_3 l_2 l_3^3}{16\{l_1(l_2 + l_3) + l_2(l_3 + \frac{3}{4} l_2)\}}$$

$$M_B = -\frac{w_2 l_2^3 - w_3 l_3^3 - 4 M_A l_2}{8(l_2 + l_3)}$$

Points of inflection :

$$ON = \frac{2 R_0}{w_1} \qquad CW = \frac{2 R_B}{w_3}.$$



(4)

$$R_0 = R_D = \tfrac{5}{16} W$$
$$R_B = \tfrac{11}{8} W$$

Bending moment :

(i) $x \leqq \frac{l}{2}$. $M_x = \frac{5}{16} W x$

(ii) $x \geqq \frac{l}{2}$ and $\leqq l$ $M_x = \frac{W}{16}(8l - 11x)$

$$M_0 = M_D = O$$
$$M_B = -\tfrac{3}{16} W l$$

Equations to elastic line :

(i) $x \leqq \frac{l}{2}$

$$y = \frac{W}{96 EI}\left(3 l^2 x - 5 x^3\right)$$

(ii) $x \geqq \frac{l}{2}$ and $\leqq l$

$$y = \frac{W}{96 EI}\left(11 x^3 - 24 l x^2 + 15 l^2 x - 2 l^3\right)$$

Maximum deflection : $x < \frac{l}{2}$

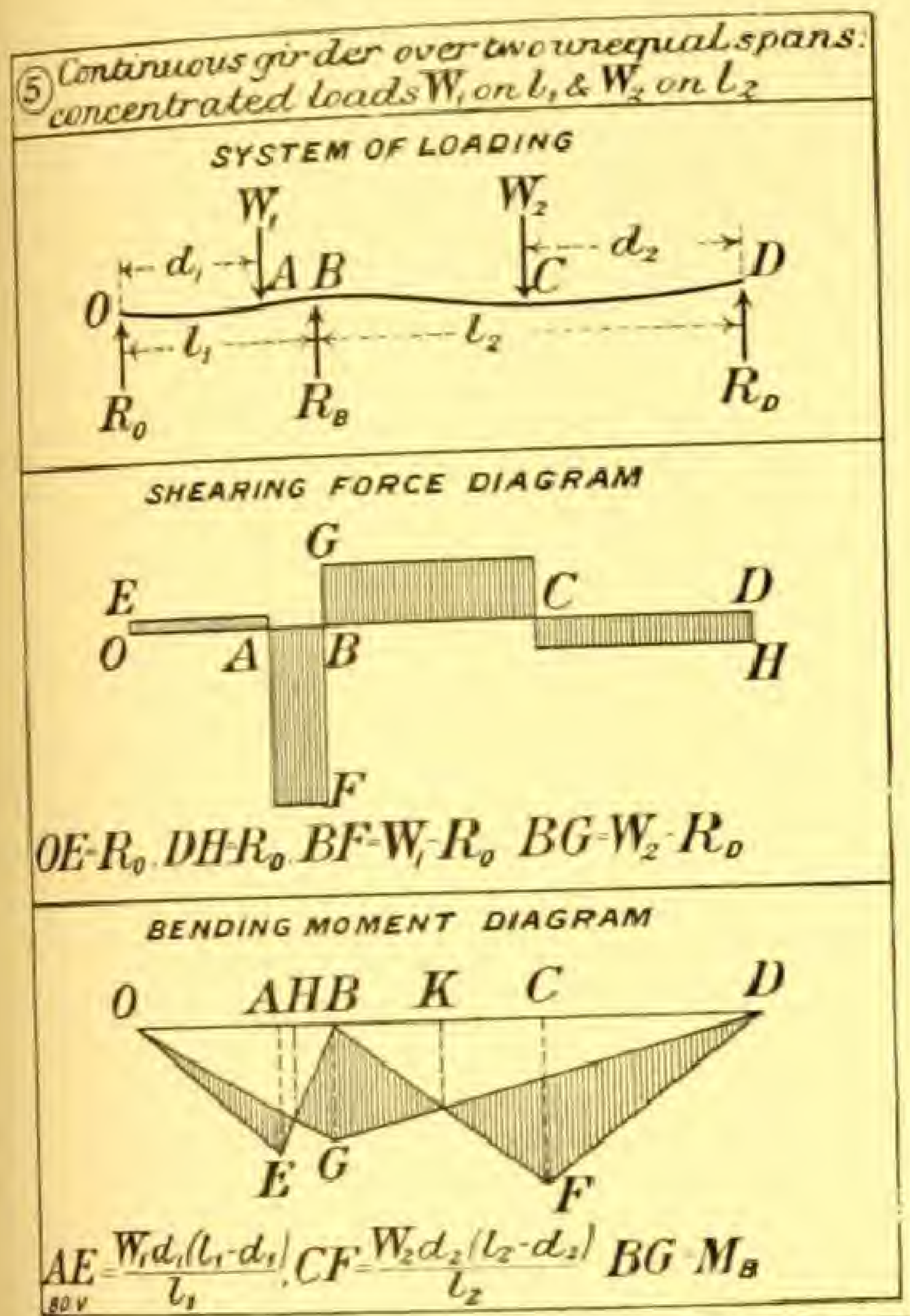
$\delta_{max.} = .0093 \frac{W l^3}{EI}$ at point where $x = .447 l$

Deflection at C $= \frac{7}{768}\frac{W l^3}{EI}$

Points of inflection occur at H and K, where $OH = DK = \frac{8}{11} l$.

l, l_1, l_2, l_3 = Spans. W, W_1, W_2, W_3 = Concentrated Loads. w, w_1, w_2, w_3 = Loads per Foot.
M_O, M_A, M_B Bending Moments at O, A, B. R_O, R_A, R_B Reactions at O, A, B.
E = Modulus of Elasticity. I = Moment of Inertia.

All expressions involve I, and therefore apply only to beams having a constant Moment of Inertia.



(5)

$$R_O = \frac{M_B + W_1(l_1 - d_1)}{l_1}$$

$$R_B = W_1 + W_2 - (R_O + R_D)$$

$$R_D = \frac{M_B + W_2(l_2 - d_2)}{l_2}$$

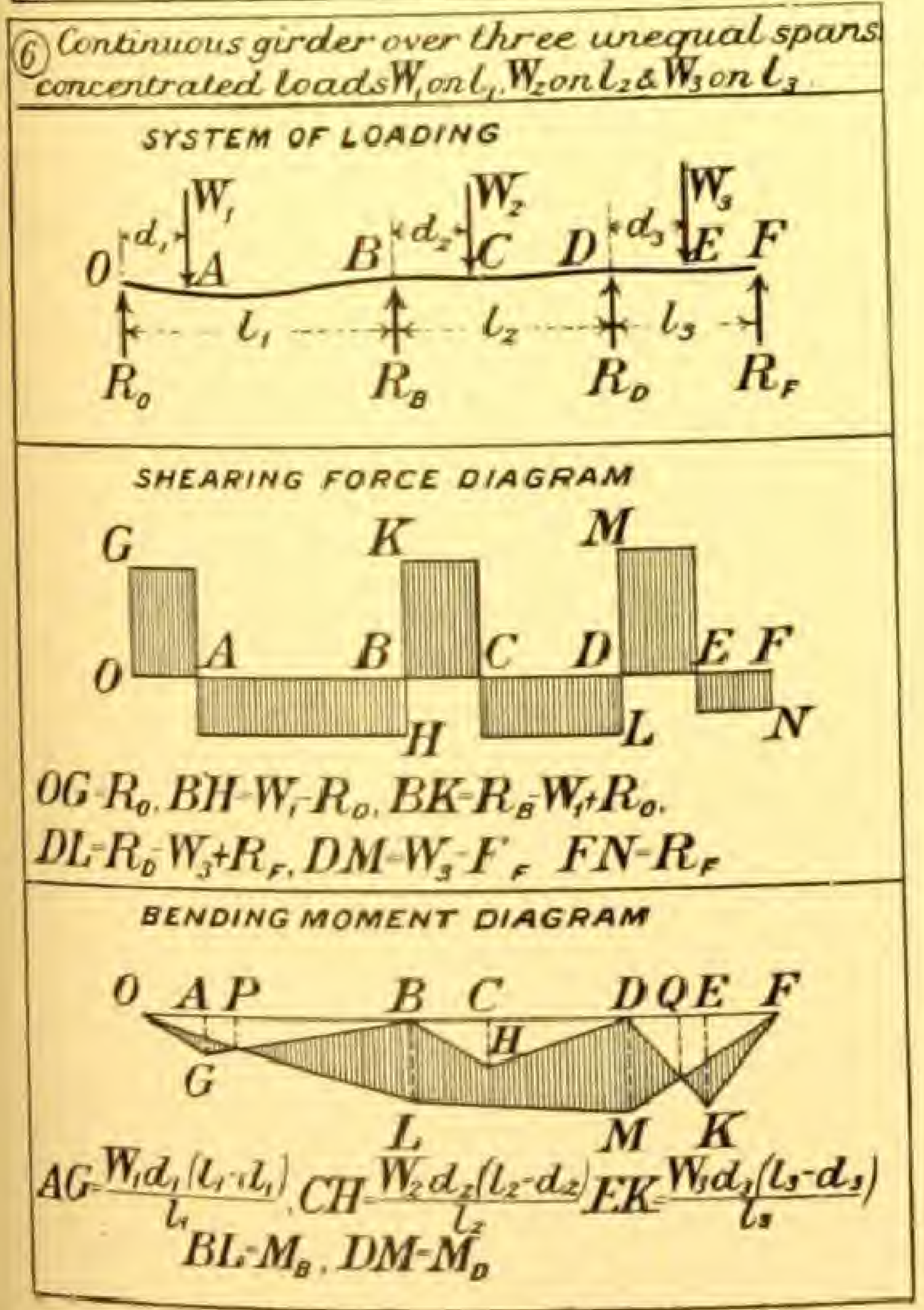
$$M_O = M_D = O$$

$$M_B = -\frac{W_1 \frac{d_1}{l_1}\left(l_1^2 - d_1^2\right) + W_2 \frac{d_2}{l_2}\left(l_2^2 - d_2^2\right)}{2(l_1 + l_2)}$$

Points of inflection occur at H and K,

where $OH = \dfrac{W_1 d_1}{W_1 - R_O}$

and $DK = \dfrac{W_2 d_2}{W_2 - R_D}$.



(6)

$$R_O = \frac{W_1(l_1 - d_1) + M_B}{l_1}$$

$$R_B = -\frac{W_1(l_1 - d_1) + M_B}{l_1} + W_1 + \frac{W_2(l_2 - d_2) - M_B + M_D}{l_2}$$

$$R_D = -\frac{W_3 d_3 + M_D}{l_3} + W_3 + \frac{W_2 d_2 + M_B - M_D}{l_2}$$

$$R_F = \frac{W_3 d_3 + M_D}{l} \qquad M_O = M_F = O$$

$$M_B = \frac{\dfrac{2W_1 d_1}{l_1}\left(l_2 + l_3\right)\left(d_1^2 - l_1^2\right) - \dfrac{W_2 d_2}{l^2}\left(l_2 - d_2\right) \times \left(3l_2^2 + 4l_2 l_3 - 3l_2 d_2 - 2l_3 d_2\right) + \dfrac{W_3 d_3 l_2}{l_3} + \left(2l_3^2 - 3l_3 d_3 + d_3^2\right)}{4(l_1 + l_2)(l_2 + l_3) - l_2^2}$$

$$M_D = -\frac{\dfrac{W_2 d_2}{l_2}\left(l_2^2 - d_2^2\right) + \dfrac{W_3 d_3}{l_3}\left(2l_3^2 - 3l_3 d_3 + d_3^2\right) + M_B l_2}{2(l_2 + l_3)}$$

Special Cases.

Inclined Cantilevers and Beams.

NOTE.—See also Conventions on page 281.

l = Span. W = Total Load. w = Load per Foot Horizontal.

M_O, M_A, M_B, M_x, Bending Moments at O, A, B, Section Distant x from O.

E = Modulus of Elasticity. R_O, R_A, R_B, Reactions at O, A, B.

δ_A, δ_B, Deflection at A, B. I = Moment of Inertia. A_1 = Area of Cross Section.

Z = Modulus of Section $= \frac{I}{y_1}$. (y_1 = Distance of Extreme Fibre from Neutral Axis).

f_1 = Direct Stress due to External Load.

f_2 = Direct Stress due to Bending $= \pm \frac{My}{I} \left(\begin{array}{l} + \text{ for Compression Fibre} \\ - \text{ for Tension Fibre} \end{array} \right)$.

$f_{max.}$ = Maximum Direct Stress $= f_1 \pm$ Max. Value of f_2.

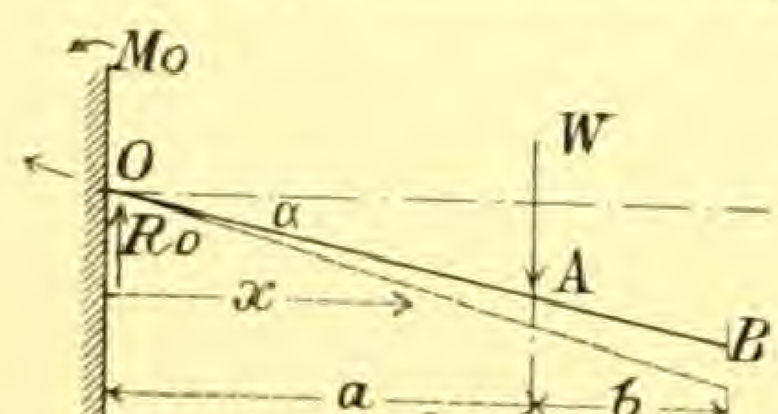
Expressions involving I apply only to beams having a constant Moment of Inertia.

α is in Degrees when Sin α or Tan α

All Deflections, y or δ, are measured from a Horizontal Line through the Origin O.

NOTE.—The Bending Moment and Shearing Force Diagrams for the following are similar in each case to the corresponding case with supports on the same levels. Where there is a parabola it will be oblique. The simplest way to draw it is first to construct a parabola in the ordinary way on the actual length of the beam OA or OB. At any section transfer the ordinate length measured at right angles to the beam to a vertical ordinate. The oblique parabola will be obtained by drawing through such points: see, for example, parabola OHA, Case 9.

(1) *Cantilever at Angle α to Horizontal. Concentrated Load W at any point.*



See diagrams Case (1) Page 282

$R_O = W$

Bending moment:

$x \leqq a \quad . \quad M_x = W(x - a)$

$x \geqq a \quad . \quad M_x = O \quad . \quad M_O = -Wa.$

Equations to elastic line:

(i) $x \leqq a \quad . \quad y = \frac{W}{6\,EI}\left[-x^3 + 3\,ax^2\right] + x \tan \alpha$

(ii) $x \geqq a \quad . \quad y = \frac{W}{6\,EI}\left[-a^3 + 3a^2x\right] + x \tan \alpha$

$$\delta_A = \frac{Wa^3}{3\,EI} + a \tan \alpha$$

$$\delta_B = \frac{Wa^2}{6\,EI}(2a + 3b) + l \tan \alpha.$$

Stress: $x < a$

$f_1 = -\frac{W \sin \alpha}{A_1} \quad . \quad f_2 = \pm \frac{W}{Z}(x - a) \quad . \quad f_{max.} = W\left(-\frac{\sin \alpha}{A_1} \pm \frac{a}{Z}\right)$ at O.

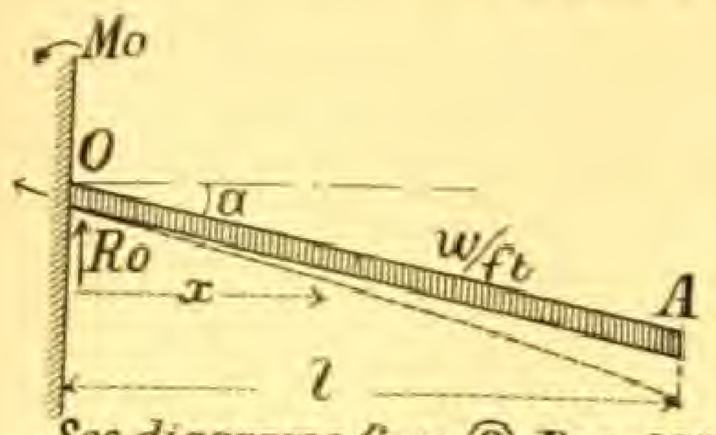
(ALL CASES.) NOTE. — (i) The deformed elastic line is in all cases shown dotted.

(ii) Z is for a *normal* section at horizontal distance x.

(iii) When several values of $f_{max.}$ are given, take the greatest for absolute maximum stress.

(iv) As all measurements are here taken horizontally, and w is a horizontal intensity, the expressions are similar to those already given for supports on same level (pages 281-300).

(2) *Cantilever at Angle α to Horizontal. Uniformly Distributed Load w per Foot Horizontal.*



See diagrams Case ② Page 282

$$R_0 = wl$$

Bending moment:

$$x \leqq l \quad . \quad M_x = -\frac{w}{2}\left(x - l\right)^2$$

$$M_0 = -\frac{wl^2}{2}.$$

Equation to elastic line:

$$x \leqq l$$

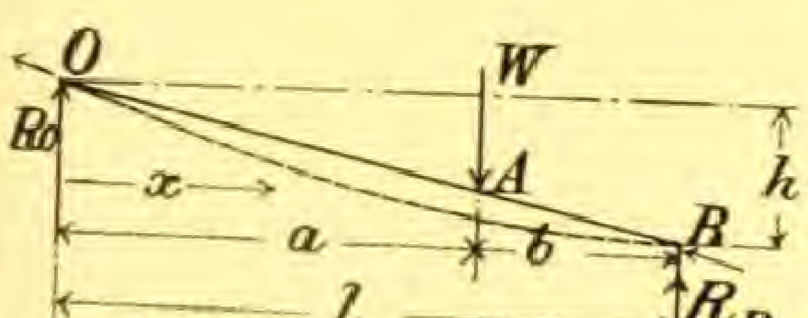
$$y = \frac{wx^2}{24\,EI}\left(x^2 - 4\,lx + 6\,l^2\right) + x\tan\alpha$$

$$\delta_A = \frac{wl^4}{8\,EI} + l\tan\alpha$$

Stress:

$$f_1 = -w\left(l - x\right)\frac{\sin\alpha}{A_1} \quad . \quad f_2 = \pm\frac{w}{2\,Z}\left(x - l\right)^2 \quad . \quad f_{max.} = wl\left(-\frac{\sin\alpha}{A_1} \pm \frac{l}{2\,Z}\right) \text{ at O.}$$

(3) *Beam simply supported at Different Levels. Concentrated Load* W.



See diagrams Case 5 Page 284

$$R_0 = \frac{Wb}{l} \quad . \quad R_B = \frac{Wa}{l}$$

Bending moment:

$$\text{(i)} \quad x \leqq a \quad . \quad M_x = \frac{Wbx}{l}$$

$$\text{(ii)} \quad x \geqq a \quad . \quad M_x = \frac{Wa}{l}\left(l - x\right)$$

$$M_A = \frac{Wba}{l}.$$

Equations to elastic line:

$$\text{(i)} \quad x \leqq a \quad . \quad y = \frac{Wb}{6\,EI\,l}\left[-x^3 + ax\,(a + 2\,b)\right] + \frac{hx}{l}$$

$$\text{(ii)} \quad x \geqq a \quad . \quad y = \frac{W}{6\,EI\,l}\left[-bx^3 + l\,(x - a)^3 + abx\,(a + 2\,b)\right] + \frac{hx}{l}$$

Maximum deflection:

$$\text{(i)} \quad a \geqq b \quad . \quad \delta_{max.} \text{ occurs at } x = \sqrt{\frac{a}{3}\left(a + 2\,b\right) + \frac{2\,EI\,h}{Wb}}$$

To get $\delta_{max.}$ insert x in equation (i) for y.

No turning point in curve between O and B when:

$$\text{(i)} \quad a \geqq b \quad . \quad h > \frac{3\,Wba}{2\,EI\ (a + 2\,b)}$$

$$\text{(ii)} \quad a \leqq b \quad . \quad h < \frac{Wba}{3\,EI}\left(b - a\right)$$

$$\delta_A = \frac{a}{l}\left[\frac{Wab^2}{3\,EI} + h\right]$$

$$\delta_{max.} \text{ will occur at A when } h = \frac{Wab}{3\,EI}\left(a - b\right).$$

Stress:

$$\text{(i)} \quad x < a \quad . \quad f_1 = \frac{-R_0\sin\alpha}{A_1} \quad . \quad f_2 = \pm\frac{Wbx}{lZ} \quad . \quad f_{max.} = \frac{Wb}{l}\left[\frac{-\sin\alpha}{A_1} \pm \frac{a}{Z}\right] \text{ at A}$$

$$\text{(ii)} \quad x > a \quad . \quad f_1 = \frac{R_B\sin\alpha}{A_1} \quad . \quad f_2 = \pm\frac{Wa}{lZ}\left(l - x\right) \quad . \quad f_{max.} = \frac{Wa}{l}\left[\frac{+\sin\alpha}{A_1} \pm \frac{l - a}{Z}\right] \text{ at point A}$$

SPECIAL CASES.

NOTE.—See also Conventions on page 281.

l = Span. W = Total Load. w = Load per Foot Horizontal.

M_0, M_A, M_B, M_x, Bending Moments at O, A, B, Section Distant x from O.

E = Modulus of Elasticity. R_0, R_A, R_B, Reactions at O, A, B.

δ_A, δ_B, Deflection at A, B. I = Moment of Inertia. A_1 = Area of Cross Section.

Z = Modulus of Section = $\frac{I}{y_1}$. (y_1 = Distance of Extreme Fibre from Neutral Axis).

f_1 = Direct Stress due to External Load.

f_2 = Direct Stress due to Bending = $\pm \frac{My}{I} \left(\begin{array}{l} + \text{ for Compression Fibre} \\ - \text{ for Tension Fibre} \end{array} \right)$.

$f_{max.}$ = Maximum Direct Stress = $f_1 \pm$ Max. Value of f_2.

Expressions involving I apply only to beams having a constant Moment of Inertia.

α is in Degrees when Sin α or Cos α.

All Deflections, y or δ, are measured from a Horizontal Line through the Origin O.

NOTE.—The Bending Moment and Shearing Force Diagrams for the following are similar in each case to the corresponding case with supports on the same levels. Where there is a parabola it will be oblique. The simplest way to draw it is first to construct a parabola in the ordinary way on the actual length of the beam OA or OB. At any section transfer the ordinate length measured at right angles to the beam to a vertical ordinate. The oblique parabola will be obtained by drawing through such points: see, for example, parabola OHA, Case 9.

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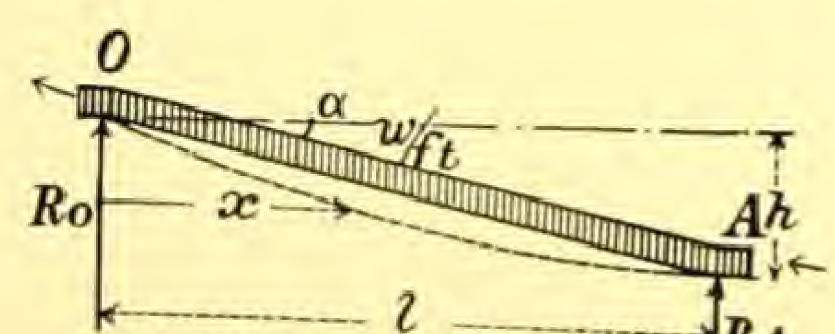
(4) *Beam simply supported at Different Levels. Uniformly Distributed Load w per Foot Horizontal.*



See diagrams Case (6) Page 284

Total load W = wl . $R_0 = R_A = \frac{wl}{2}$

Bending moment:

$$M_x = \frac{w}{2}\left(lx - x^2\right)$$

$$M_{max.} = \frac{wl^2}{8} \text{ at centre.}$$

Equation to elastic line:

$$y = \frac{w}{24\,EI}\left[x^4 - 2\,lx^3 + l^3x\right] + \frac{hx}{l}.$$

Point of maximum deflection is at the value of x in

$$4\,x^3 - 6\,lx^2 + l^3 + \frac{24\,EI\,h}{wl} = 0.$$

There will be no turning point between O and A unless the value of x is + and $\leqq l$.

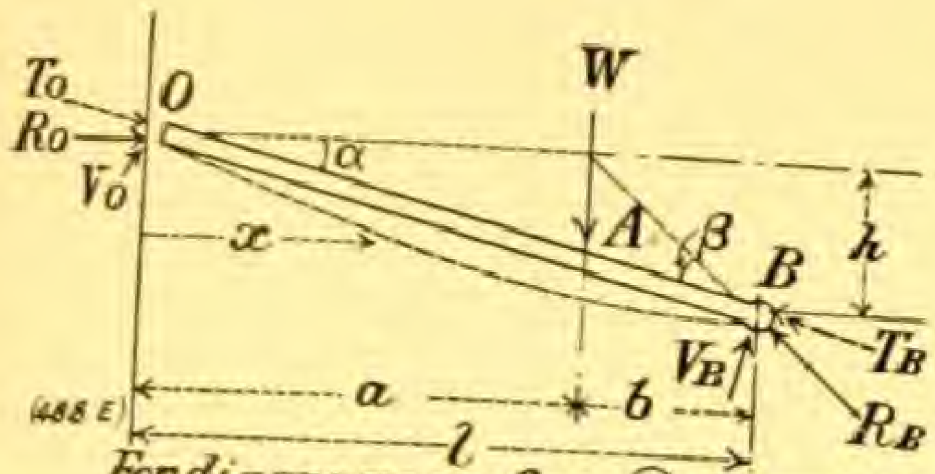
$\delta_{max.}$ will just occur at A when $h = \frac{wl^4}{24\,EI}$.

Stress:

$$f_1 = \frac{w \sin \alpha}{A_1}\left(x - \frac{l}{2}\right) \qquad . \qquad f_2 = \pm \frac{w}{2\,Z}\left(lx - x^2\right)$$

$$f_{max.} = \pm \frac{wl \sin \alpha}{2A_1} \; ; \; \left(+ \text{ at O}, - \text{ at A}\right) \qquad f_{max.} = \pm \frac{wl^2}{8Z} \text{ at centre.}$$

(5) *Inclined Beam, Pin Jointed at* B *and resting against Wall at* O. *Concentrated Load* W.



For diagrams see Case (5) Page 284.

$$R_0 = \frac{Wb}{h} \quad . \quad V_0 = \frac{Wb \sin \alpha}{h}$$

$$V_B = \frac{Wa \cos \alpha}{l} \quad . \quad R_B = W\frac{\sqrt{b^2 + h^2}}{h}.$$

NOTE.—For Bending Moment, Equations to Elastic Line, Deflection at A, Point of Maximum Deflection, Maximum Deflection, Conditions for Maximum Deflection to occur at A, and Conditions for no Turning Point in Curve between O and B, use the Equations of Case (3).

Stress:

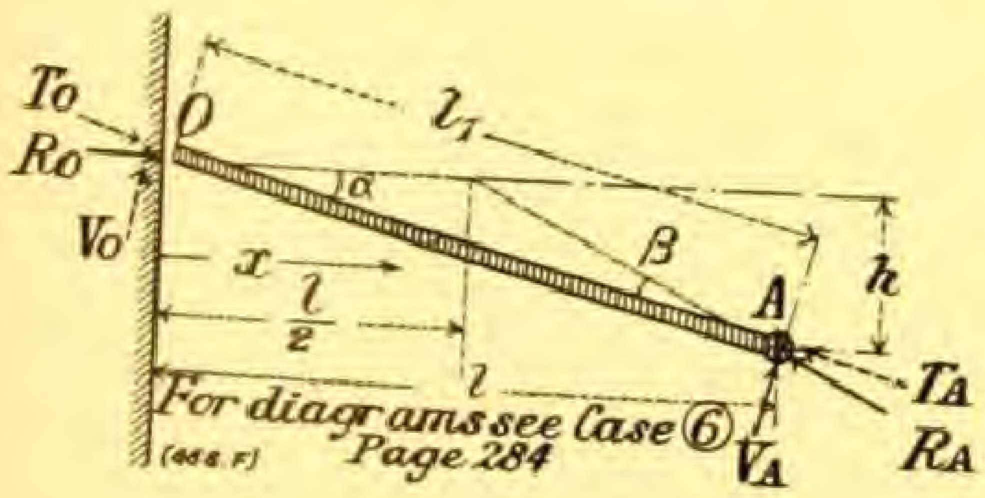
(i) $x < a$. $f_1 = \dfrac{R_0 \cos \alpha}{A_1}$. $f_2 = \pm \dfrac{Wbx}{lZ}$

$$f_{max.} = \frac{R_0 \cos \alpha}{A_1} \text{ at O} \quad . \quad f_{max.} = \frac{R_0 \cos \alpha}{A_1} \pm \frac{Wba}{lZ} \text{ at A.}$$

(ii) $x > a$. $f_1 = \dfrac{R_B \cos \beta}{A_1} = \dfrac{R_0 \cos \alpha + W \sin \alpha}{A_1}$. $f_2 = \pm \dfrac{Wa}{lZ}\left(l - x\right)$

$$f_{max.} = \frac{R_0 \cos \alpha + W \sin \alpha}{A_1} \pm \frac{Wab}{lZ} \text{ at A} \quad . \quad f_{max.} = \frac{R_0 \cos \alpha + W \sin \alpha}{A_1} \text{ at B.}$$

(6) *Inclined Beam, Pin Jointed at* A *and resting against Wall at* O. *Uniformly Distributed Load* w *per Foot Horizontal.*



For diagrams see Case (6) Page 284

$$R_0 = \frac{wl^2}{2h} \quad . \quad T_0 = \frac{wl^3}{2hl_1}$$

$$R_A = \frac{wl}{2h}\sqrt{l^2 + 4h^2}.$$

NOTE.—For Bending Moment, Maximum Bending Moment, Equations to Elastic Line, Point of Maximum Deflection, Maximum Deflection, and Conditions for Maximum Deflection to occur at A, use the Equations of Case (4).

Stress:

$$f_1 = \frac{R_0 \cos \alpha + wx \sin \alpha}{A_1} \quad . \quad f_2 = \pm \frac{wx}{2Z}\left(l - x\right)$$

$$f_{max.} = \frac{R_0 \cos \alpha}{A_1} \text{ at O.} \quad . \quad f_{max.} = \frac{R_0 \cos \alpha + wl \sin \alpha}{A_1} \text{ at A.}$$

$$f_{max.} = \frac{1}{A_1}\left(R_0 \cos \alpha + \frac{wl}{2} \sin \alpha\right) \pm \frac{wl^2}{8Z} \text{ at centre.}$$

SPECIAL CASES.

NOTE.—See also Conventions on page 281.

l = Span. W = Total Load. w = Load per Foot Horizontal.

M_O, M_A, M_B, M_x, Bending Moments at O, A, B, Section Distant x from O.

E = Modulus of Elasticity. R_O, R_A, R_B, Reactions at O, A, B.

δ_A, δ_B, Deflection at A, B. I = Moment of Inertia. A_1 = Area of Cross Section.

Z = Modulus of Section = $\frac{I}{y_1}$. (y_1 = Distance of Extreme Fibre from Neutral Axis.)

f_1 = Direct Stress due to External Load.

$$f_2 = \text{Direct Stress due to Bending} = \pm \frac{My}{I}\left(\begin{array}{l}+\text{ for Compression Fibre}\\ -\text{ for Tension Fibre}\end{array}\right).$$

$f_{max.}$ = Maximum Direct Stress = $f_1 \pm$ Max. Value of f_2.

Expressions involving I apply only to beams having a constant Moment of Inertia.

α is in Degrees when Sin α or Tan α.

All Deflections, y or δ, are measured from a Horizontal Line through the Origin O.

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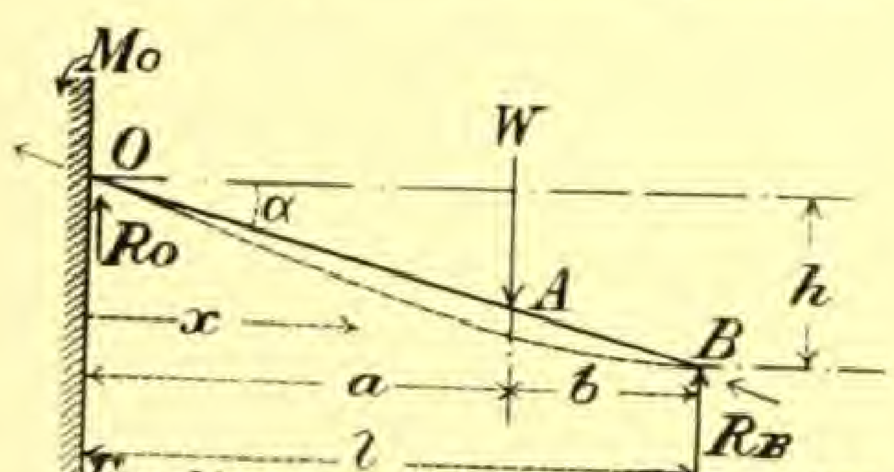
(7) *Inclined Beam, fixed at* O *and simply supported at* B. *Concentrated Load* W.



For diagrams see Case 20 Page 293

$$R_O = \frac{1}{2l^3}\left[W(a^3 - 3a^2l + 2l^3)\right]$$

$$R_B = \frac{1}{2l^3}\left[Wa^2(2a + 3b)\right].$$

Bending moment:

(i) $x \leqq a$. $M_x = M_O + R_O x$

(ii) $x \geqq a$. $M_x = R_B(l - x)$

$$M_O = -\frac{1}{2l^2}\left[Wab(a + 2b)\right] \qquad M_A = \frac{b}{2l^3}\left[Wa^2(2a + 3b)\right].$$

Equations to elastic line:

(i) $x \leqq a$. $y = \frac{1}{6EI}\left[-R_O x^3 - 3M_O x^2\right] + x\tan\alpha$

(ii) $x \geqq a$. $y = \frac{1}{6EI}\left[-R_O x^3 - 3M_O x^2 + W(x - a)^3\right] + x\tan\alpha$

$$\delta_A = \frac{a^2}{6EI}\left[-R_O a - 3M_O\right] + a\tan\alpha$$

Point of maximum deflection occurs at

$$x = \frac{2m \pm \sqrt{4m^2 - 4n(Wa^2 + 2EI\tan\alpha)}}{2n}$$

where $m = (M_O + Wa)$. $n = (W - R_O)$.

Stress:

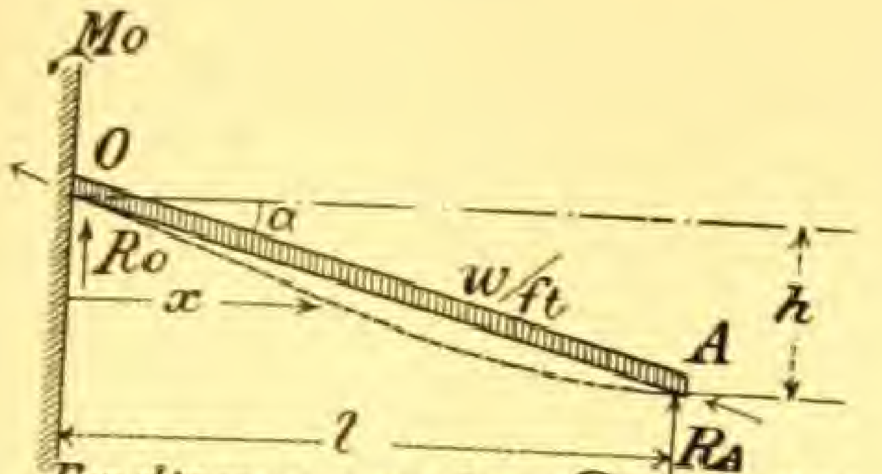
(i) $x < a$. $f_1 = -\frac{R_O \sin\alpha}{A_1}$. $f_2 = \pm\frac{M_O + R_O x}{Z}$

$$f_{max.} = -\frac{R_O\sin\alpha}{A_1} \pm \frac{M_A}{Z} \text{ or } \frac{M_O}{Z} \text{ at A or O respectively.}$$

(ii) $x > a$. $f_1 = \frac{R_B \sin\alpha}{A_1}$ $f_2 = \pm\frac{R_B(l - x)}{Z}$. $f_{max.} = \frac{R_B\sin\alpha}{A_1} \pm \frac{M_A}{Z}$ at A.

$$f_{max.} = \frac{R_B\sin\alpha}{A_1} \text{ at B.}$$

(8) *Inclined Beam, fixed at O and simply supported at B. Uniformly Distributed Load w per Foot Horizontal.*



For diagrams see Case (18) Page 291.
(488 K)

$$R_0 = \frac{5}{8} wl. \qquad R_A = \frac{3}{8} wl.$$

Bending moment:

$$M_x = R_A (l - x) - \frac{w}{2}(l - x)^2$$

$$M_0 = -\frac{wl^2}{8}$$

$$M_{max.}\ \text{pos.} = \frac{9}{128} wl^2, \text{ at point where } x = \frac{5}{8} l.$$

Equation to elastic line: $y = \frac{1}{24\,EI}(wx^4 - 4\,R_0 x^3 - 12\,M_0 x^2) + x \tan a.$

Point of maximum deflection is the value of x in $wx^3 - 3\,R_0 x^2 - 6\,M_0 x + 6\,EI \tan a = O.$

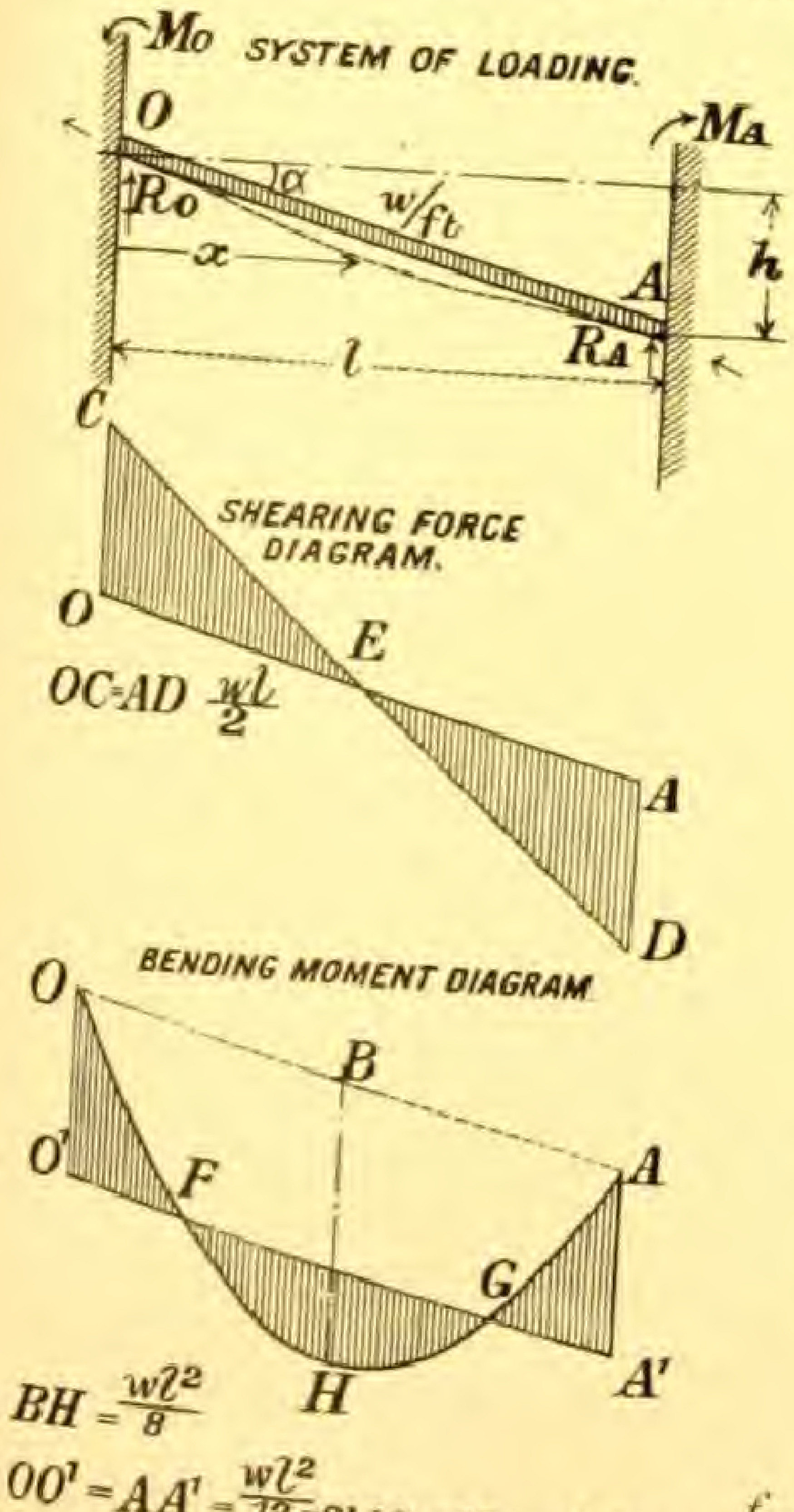
Point of inflection:

$$x = \frac{l}{4}.$$

Stress: $f_1 = \frac{-R_0 \sin a + wx \sin a}{A_1}$. $f_2 = \pm \frac{M_x}{Z}$. $f_{max.} = \frac{-R_0 \sin a}{A_1} \pm \frac{M_0}{Z}$ at O.

$f_{max.} = \frac{R_A \sin a}{A_1}$ at A. $f_{max.} = \pm M_{max.\ pos.}$ at $x = \frac{R_0}{w} = \frac{5\,l}{8}$.

(9) *Inclined Beam fixed at both ends. Uniformly Distributed Load w per Foot Horizontal.*



$BH = \frac{wl^2}{8}$

$OO' = AA' = \frac{wl^2}{12}$ = Shift of Base L due to fixed ends.

OHA is an oblique parabola, Axis BH, Vertex H.
(488 K)

$$R_0 = \frac{wl}{2} \qquad R_A = \frac{wl}{2}.$$

Bending moment:

$$M_x = M_0 + R_0 x - \frac{wx^2}{2}$$

$$M_0 = -\frac{wl^2}{12}$$

$$M_A = -\frac{wl^2}{12}$$

$$M_{max.}\ \text{pos.} = \frac{wl^2}{24} \text{ at } x = \frac{l}{2}.$$

Equation to elastic line:

$$y = \frac{1}{24\,EI}\left[wx^4 - 4\,R_0 x^3 - 12\,M_0 x^2\right] + x \tan a.$$

Point of maximum deflection is at the value of x in

$$wx^3 - 3\,R_0 x^2 - 6\,M_0 x + 6\,EI \tan a = O.$$

Point of inflection:

$$x = 0.211\,l \text{ or } 0.789\,l$$

Stress:

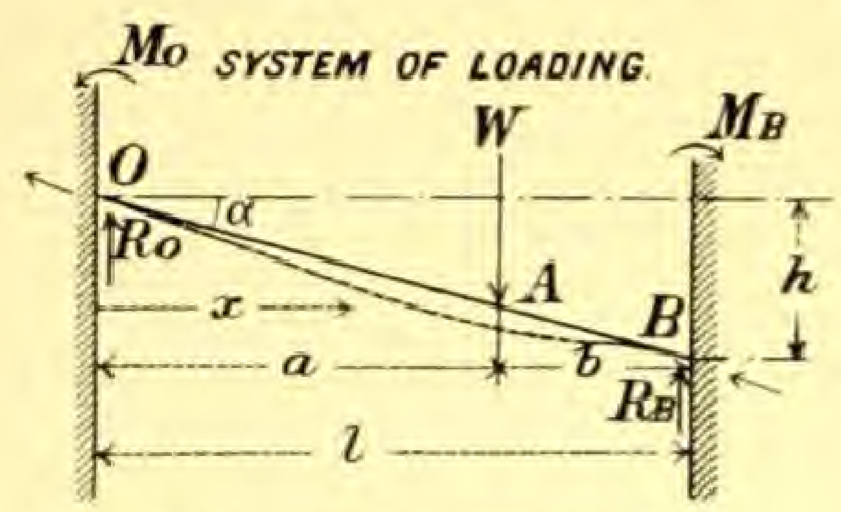
$$f_1 = \frac{-R_0 \sin a + wx \sin a}{A_1} \qquad f_2 = \pm \frac{M_x}{Z}$$

$$f_{max.} = \frac{-R_0 \sin a}{A_1} \pm \frac{M_0}{Z} \text{ at O.} \qquad f_{max.} = \frac{R_A \sin a}{A_1} \pm \frac{M_A}{Z} \text{ at A.}$$

$$f_{max.} = \pm M_{max.\ pos.} \text{ at } x = \frac{R_0}{w} = \frac{l}{2}.$$

SPECIAL CASES.—Refer to Heading on page 310.

(10) *Inclined Beam, fixed at both ends. Concentrated Load* W *at any point*



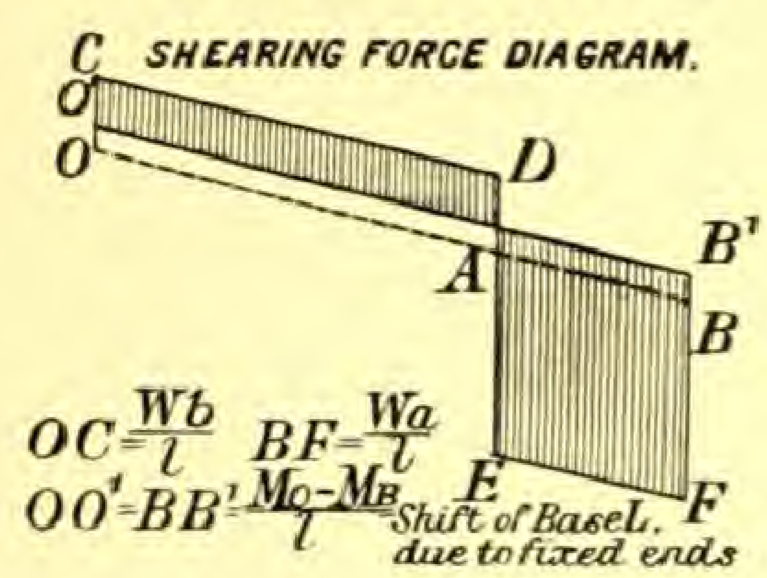
$$R_O = \frac{1}{l^3}\Big[\, Wb^2\,(3\,a + b)\,\Big]$$

$$R_B = \frac{1}{l^3}\Big[\, Wa^2\,(a + 3\,b)\,\Big]$$

$$M_O = -\frac{1}{l^2}\Big[\, Wab^2\,\Big]$$

$$M_B = -\frac{1}{l^2}\Big[\, Wa^2b\,\Big]$$

$$M_A = \frac{1}{l^3}\Big[\, 2\,Wa^2b^2\,\Big]$$



$$AG = \frac{Wab}{l}$$

Bending moment :

(i) $x \leqq a$. $M_x = M_O + R_O x$

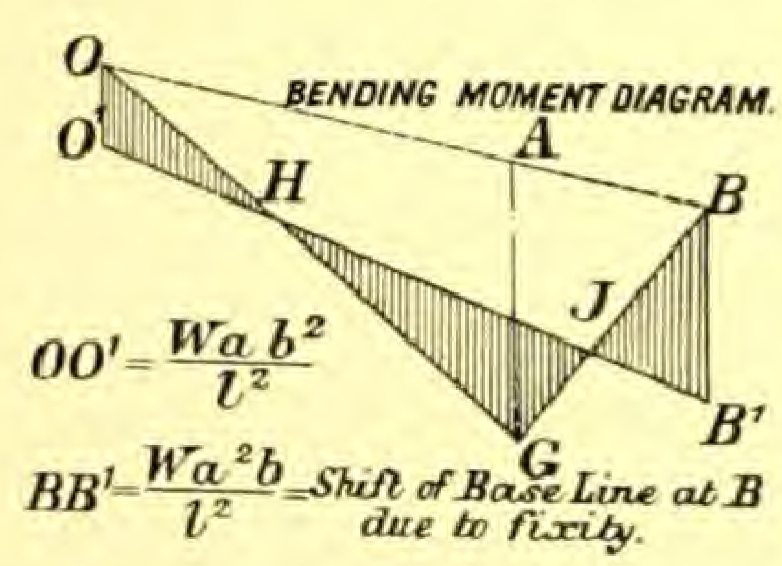
(ii) $x \geqq a$. $M_x = M_O + R_O x - W\,(x - a)$

Equations to elastic line :

(i) $x \leqq a$. $y = \dfrac{1}{6\,EI}\Big[-R_O x^3 - 3\,M_O x^2\Big] + x \tan\alpha$

(ii) $x \geqq a$. $y = \dfrac{1}{6\,EI}\Big[-R_O x^3 - 3\,M_O x^2 + W(x-a)^3\Big] + x \tan\alpha$

$$\delta_A = \frac{Wa^3b^3}{3\,EI\,l^3} + a \tan\alpha.$$



Point of maximum deflection :

(i) $a \geqq b$ $\qquad x = \dfrac{-M_O \pm \sqrt{M_O^2 + 2\,R_O\,EI\tan\alpha}}{R_O}$

(ii) $a \leqq b$. $\qquad x = \dfrac{-m \pm \sqrt{m^2 + n\,(Wa^2 + 2\,EI\tan\alpha)}}{n}$

Where $m = (M_O + Wa)$. $n = (R_O - W)$.

Points of inflection :

(i) $x < a$. $x = \dfrac{al}{3a + b}$.

(ii) $x > a$. $x = \dfrac{l\,(a + 2b)}{a + 3b}$.

Stress : (i) $x < a$

$f_1 = -\dfrac{R_O \sin\alpha}{A_1}$. $f_2 = \pm\dfrac{M_O + R_O x}{Z}$. $f_{max.} = -\dfrac{R_O\sin\alpha}{A_1} \pm \left\{\dfrac{M_O}{Z} \text{ or } \dfrac{M_O + R_O a}{Z}\right\}$ at O or A respectively.

(ii) $x > a$

$f_1 = \dfrac{R_B \sin\alpha}{A_1}$. $f_2 = \pm\dfrac{M_B + R_B}{Z}\Big(l - x\Big)$ $\quad f_{max.} = \dfrac{R_B\sin\alpha}{A_1} \pm \left\{\dfrac{M_B}{Z} \text{ or } \dfrac{M_A}{Z}\right\}$ at B or A respectively.

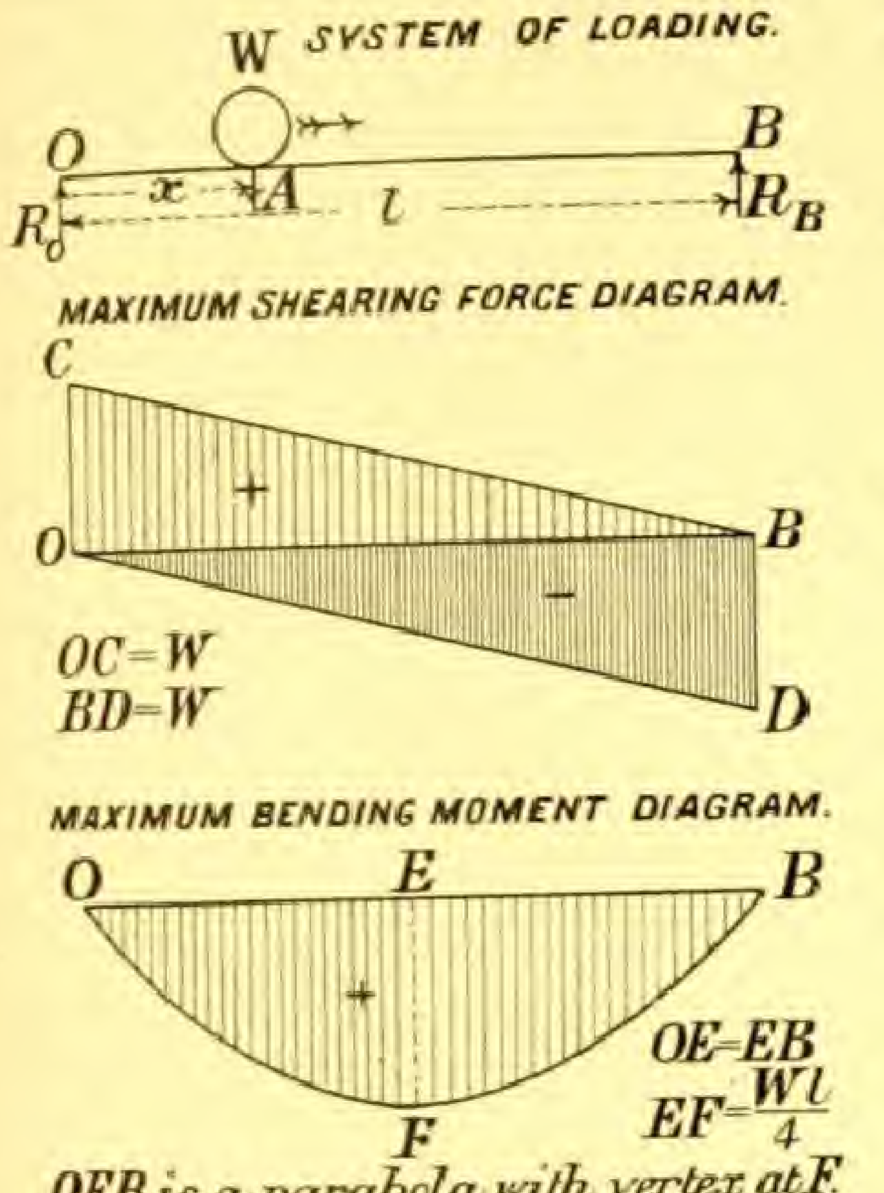
Moving Loads.

l = Span. W, W_1, W_2 = Concentrated Loads. w = Load per Foot.

R_o, R_B, R_C, R_D = Reactions at O, B, C, D. S_o, S_B, S_X = Shearing Forces at O, B and Section Distant x.

M_E, M_P, M_X = Bending Moments at E, P and Section Distant x.

x in all cases is measured from the left support towards the right.



OFB is a parabola with vertex at F.
(438 A)

(1) *Single Concentrated Load W.*

Shearing Force:

The greatest positive shearing force at any section occurs when the load is just to the right of the section,

$$\text{Max. pos. } S_X = \frac{W(l-x)}{l}$$

The greatest negative shearing force occurs when the load is just to the left of the section,

$$\text{Max. pos. } S_X = -\frac{Wx}{l}$$

$\therefore$ when $x = 0$ Max. $S_o = R_o = W$

when $x = l$ Max. $S_B = -R_B = -W$

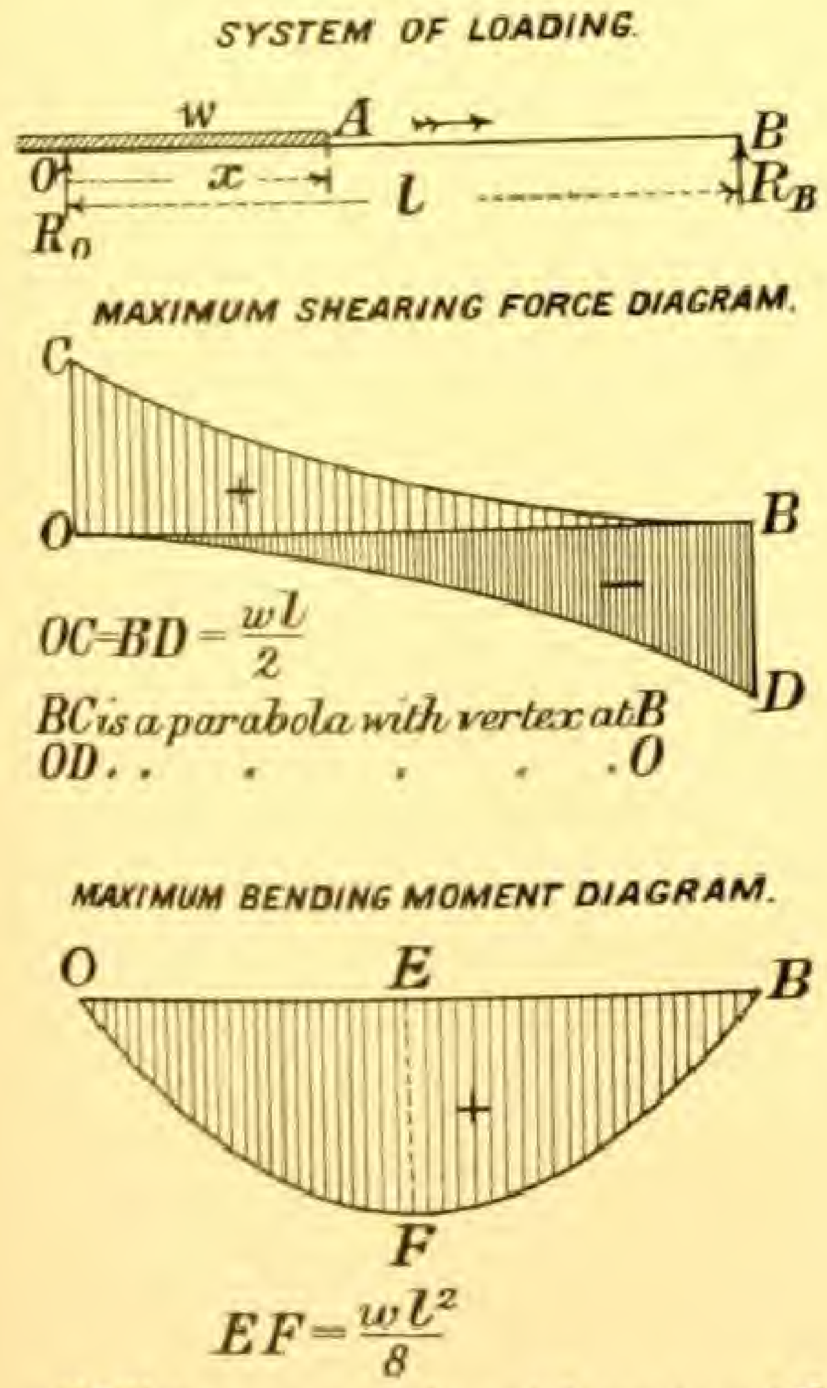
Bending Moment:

The maximum bending moment at any section occurs when the load is over the section,

$$\text{Max. } M_X = \frac{Wx}{l}\left(l - x\right) \text{ the equation to a parabola}$$

$$\text{Absolute Max. } M_X = M_E = \frac{Wl}{4}$$

Note.—In using the Shearing Force Diagram, the shear at any section is the greater of the two values (positive or negative), given by the diagram, not their sum.



BC is a parabola with vertex at B
OD . . . . . . O

OFB is a parabola with vertex at F.
(438 C)

(2) *Uniformly Distributed Load, w per foot run, of length greater than the span.*

Shearing Force:

Max. positive at any section occurs when the right only is fully loaded up to that section.

$$\text{Max. pos. } S_X = R_o = w\frac{(l-x)^2}{2l} \left(\begin{array}{c}\text{load coming on}\\ \text{from right}\end{array}\right)$$

$$S_o = \frac{wl}{2}$$

Max. neg. shear when the left only is fully loaded.

$$\text{Max. neg. } S_X = -R_B = -\frac{wx^2}{2l}$$

$$S_B = -\frac{wl}{2}$$

Bending Moment:

Max. at any section occurs when beam is fully loaded, and

$$R_o = R_B = \frac{wl}{2}$$

$$M_X = \frac{wx}{2}\left(l - x\right)$$

$$\text{Absolute max. } M_E = \frac{wl^2}{8}.$$

l = Span. W, W_1, W_2 = Concentrated Loads. w = Load per Foot.
R_O, R_B, R_C, R_D = Reactions at O, B, C, D. S_O, S_B, S_X = Shearing Forces at O, B and Section Distant x.
M_E, M_P, M_X = Bending Moments at E, P and Section Distant x.
x in all cases is measured from the left support towards the right.

---

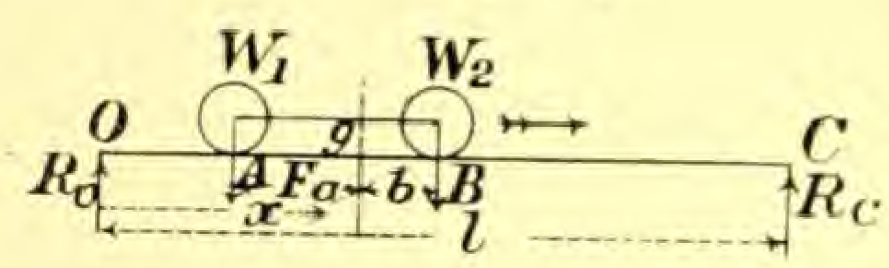
(3) *Two Concentrated Loads* W_1 *and* W_2 *at fixed distance* $(a + b)$.

SYSTEM OF LOADING.

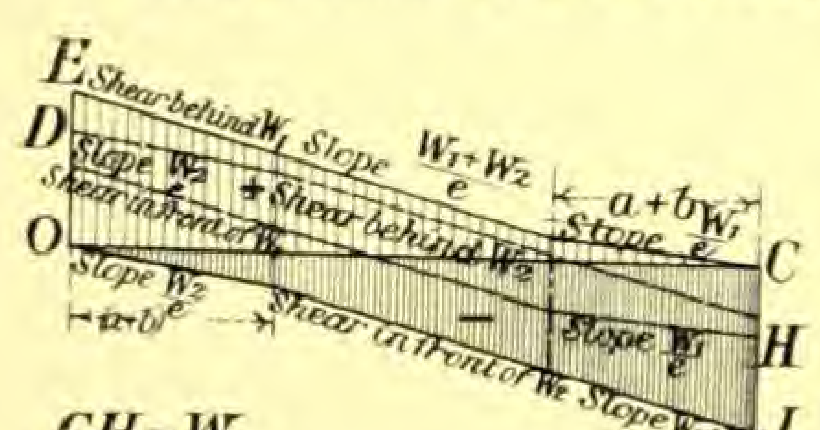
g is centre of gravity of loads.

$$a = \frac{W_2(a+b)}{W_1+W_2}$$

$$b = \frac{W_1(a+b)}{W_1+W_2}$$

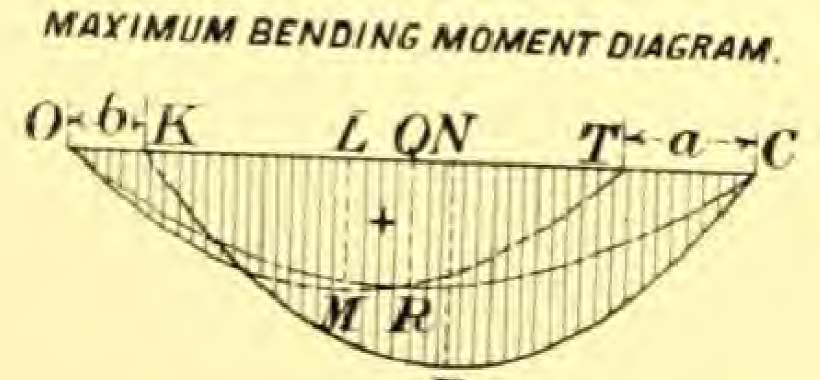


MAXIMUM SHEARING FORCE DIAGRAM.



$CH = W_1$
$OD = W_2$

MAXIMUM BENDING MOMENT DIAGRAM.



$$NP = \frac{W_1+W_2}{4l}(l-b)^2$$

$$LM = \frac{W_1+W_2}{4l}(l-a)^2$$

$$QR = \frac{W_2 l}{4}$$

KPC is a parabola with vertex at P.
OMT . . . . . M.
ORC . . . . . R.
(438 B.)

SHEARING FORCE: O F = A B.

(1) Positive Shear.

The greatest positive shearing force occurs when either W_1 or W_2 is immediately to the right of the section.

For sections between O and F, $S_X = R_O = \frac{W_1 + W_2}{l}\left(x - a\right)$

For sections between F and C, $S_X = \frac{W_1}{l}x$.

(2) Negative Shear.

The greatest negative shearing force at any section occurs when either W_1 or W_2 is immediately to the left of the section.

For sections between O and F, $S_X = -\frac{W_2}{l}x$.

For sections between F and C, $S_X = -R_C = -\frac{W_1 + W}{l}\left(x - b\right)$

This is a straight line of slope $= -\frac{W_1 + W_2}{l}$

The shear in front of W_1 = shear behind W_2, but occurs at distance $(a + b)$ behind it. Also shear immediately behind W_2 differs from that in front by the amount W_2.

BENDING MOMENT:

The maximum bending moment at any section occurs when a load is over that section.

(1) Due to W_2 over any section distant x.

$$R_C = \frac{W_1 + W_2}{l}\left(x - b\right)$$

$$M_X = \frac{W_1 + W_2}{l}\left(x - b\right)\left(l - x\right)$$

$$\text{Absolute max. } M_N = \frac{W_1 + W_2}{4l}\left(l - b\right)^2.$$

(2) Due to W_1 over any section distant x.

$$M_X = R_O x = \frac{W_1 + W_2}{l}\left(l - x - a\right)x, \text{ a parabola.}$$

$$\text{Absolute max. } M_L = \frac{W_1 + W_2}{4l}\left(l - a\right)^2$$

When $(a + b) > \frac{W_1}{W_1 + W_2}l$, a part of the diagram will be a parabola of height $\frac{W_2 l}{4}$ due to the passage of the heavier of the two loads (W_2 in this case) across the beam.

l = Span. W, W_1, W_2 = Concentrated Loads. w = Load per Foot.
R_0, R_B, R_C, R_D = Reactions at O, B, C, D. S_0, S_B, S_X = Shearing Forces at O, B and Section Distant x.
M_E, M_P, M_X = Bending Moments at E, P and Section Distant x.
x in all cases is measured from the left support towards the right

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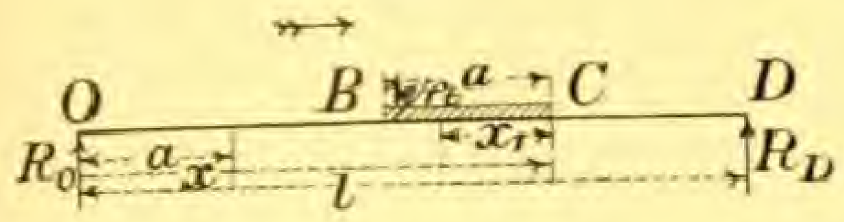
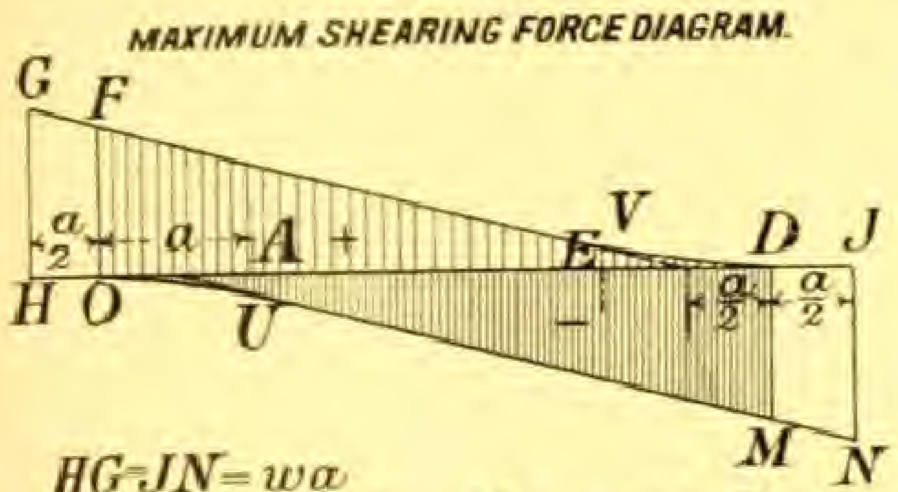
(4) *Uniformly Distributed Load, w per foot run, of length less than the span.*

SYSTEM OF LOADING

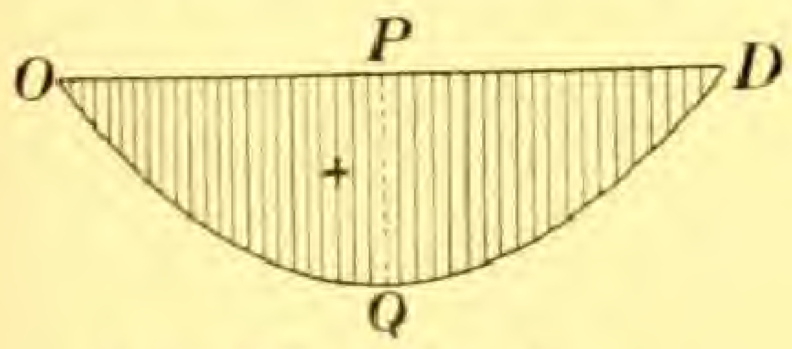


MAXIMUM SHEARING FORCE DIAGRAM.



HG=JN=wa
VD is a parabola with vertex at D.
OU . . . . . . O.

MAXIMUM BENDING MOMENT DIAGRAM.



$PQ = \frac{wa}{8}\ (2l - a)$
OQD is a parabola with vertex at Q.
(438.D.)

SHEARING FORCE:

(1) Positive Shear:

Max. positive occurs when front of load is at A

(a) load partially on (*vide* Case 3)

$$\text{Max. pos. } S_X = \frac{w\,(l - x)^2}{2\,l}$$

(b) load fully on

$$\text{Max. pos. } S_X = \frac{wa}{l}\left(l - x + \frac{a}{2}\right)$$

$$\text{Absolute max. pos. } S_0 = \frac{wa}{2\,l}\left(2\,l - a\right).$$

(2) Negative Shear:

Max. negative occurs when front of load is at D

(a) load partially on

$$\text{Max. neg. } S_X = -\frac{wx^2}{2\,l}$$

(b) load fully on

$$\text{Max. neg. } S_X = -\frac{wa}{l}\left(x - \frac{a}{2}\right)$$ the equation to a straight line of slope $-\frac{wa}{l}$ cut by O D at a distance $\frac{a}{2}$ from O

$$\text{Absolute max. neg. } S_D = -\frac{wa}{2\,l}\left(2\,l - a\right).$$

BENDING MOMENT:

The maximum at any section occurs for some position of the load over that section

$$\text{Max. } M_X = \frac{wax}{l}\left(1 - \frac{a}{2\,l}\right)\left(l - x\right)$$ and occurs when $\frac{l - x}{x} = \frac{x_1}{l - x_1}$, that is, at a section K, such that,

$$\frac{DK}{OK} = \frac{CK}{BK}$$

$$\text{Absolute max. } M_P = \frac{wa}{8}\left(2\,l - a\right).$$

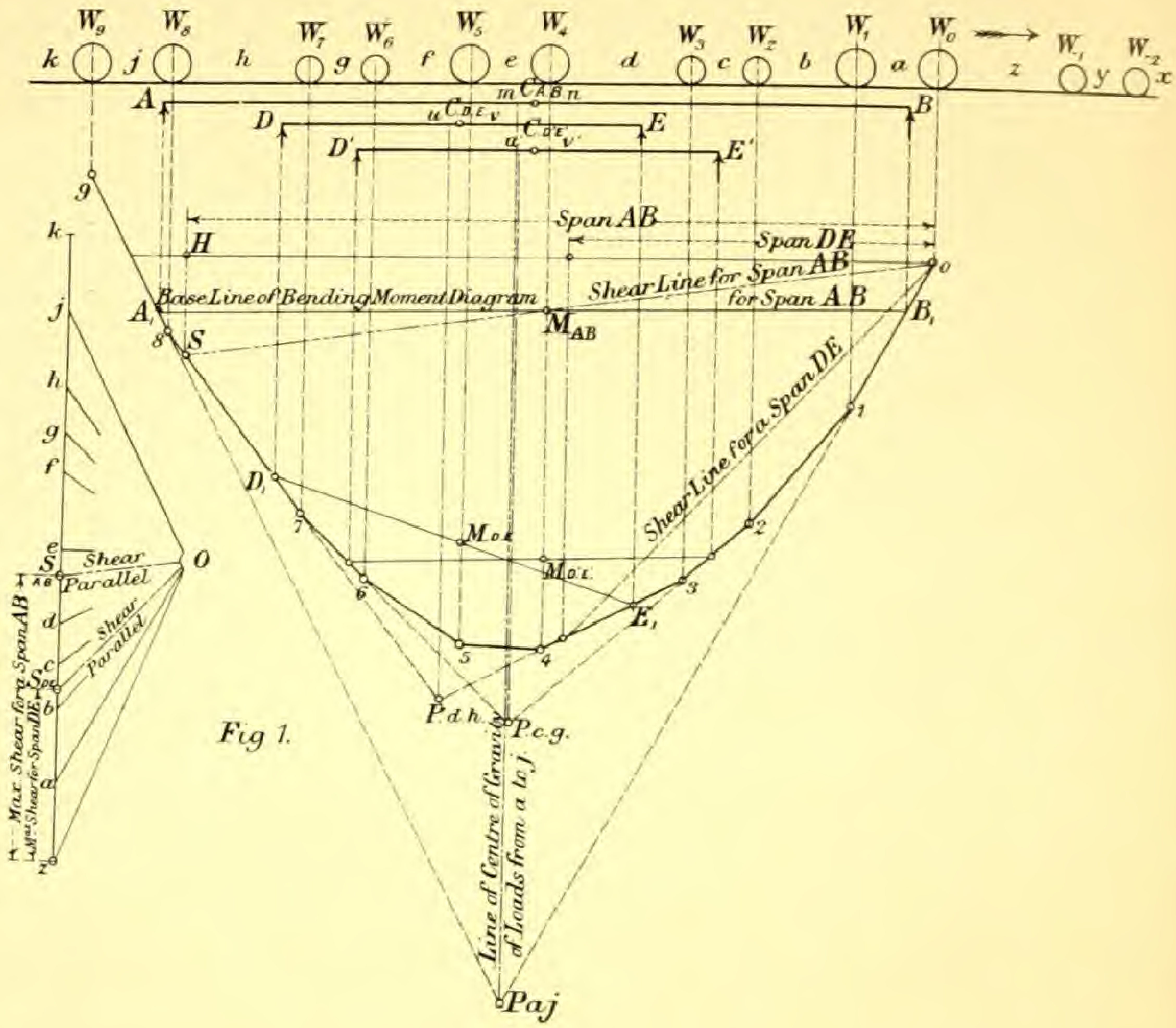
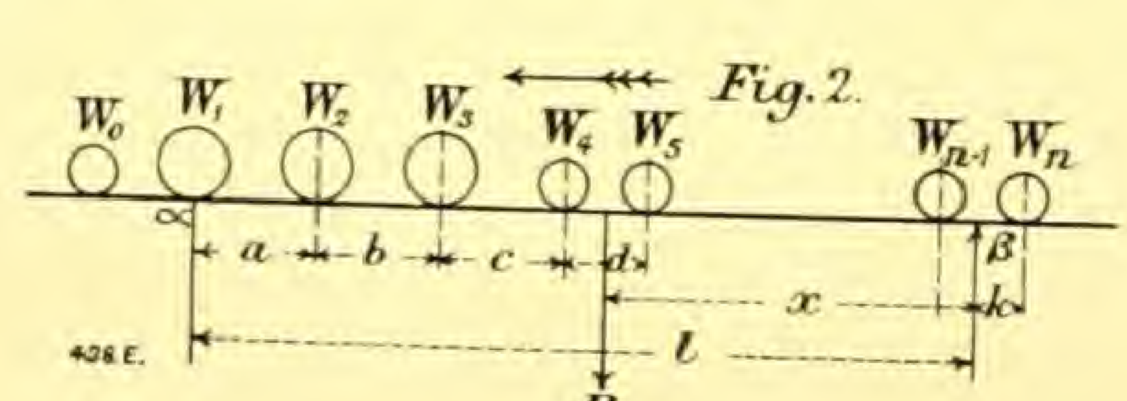


Fig 1.

Fig. 2.



Consider any system of concentrated moving loads W_{-2} to W_9 (Fig. 1) at fixed distances apart. Draw the force and link polygon for the system.

MAXIMUM BENDING MOMENT:

"The maximum bending moment under any load occurs when that load and the centre of gravity of the whole series are equidistant from the centre of the span."

For example, take a beam of span A B subject to the load system a to j or W_1 to W_8. Produce the first and last links 0 1 and 8 9 to meet in P_{aj}. Then the vertical through P_{aj} is the line of the centre of gravity of the system. For maximum bending moment under any load such as W_4, the distance $m\,n$ must be bisected in C_{AB}, which is the centre of the beam. Now place A B with centre at this point. Project down A and B to cut the polygon in $A_1\,B_1$. Join $A_1\,B_1$. Then the maximum bending moment under load W_4 is $M_{AB}\,4$.

ABSOLUTE MAXIMUM BENDING MOMENTS :

The maximum bending moment at any point along the span will take place when one or other of the wheel loads is over the point. The absolute maximum bending moment for any given span is found by making several trials. It is usually sufficient to test two, or at most three, load systems, as it occurs when the heaviest wheel or wheels are at or near the centre. Consider a span D E. Marking the span on a strip of paper, and holding it under the loads, we see that there are only two likely load systems, d to h and c to g. Taking d to h, find the line of the centre of gravity as before, and set off the beam with its centre bisecting the distance between this line and the nearest wheel load, in this case W_5. Project D E down to cut the link polygon in D_1 and E_1. Join $D_1 E_1$. Then under load W_5 we have $M_{DE}5$ as the maximum bending moment. On trying load system c to g the beam takes the position D′ E′, and the maximum bending moment is $M_{D'E'}4$, occurring under wheel W_4. This comes out less than $M_{DE}5$, so that for span D E the absolute maximum bending moment is $M_{DE}5$.

MAXIMUM SHEARING FORCE :

Consider the load system in Fig. 2, moving from right to left over a beam $\alpha\beta$ of span l. Let R be the resultant of all the loads on the beam, that is, their sum. It can easily be shown that if

$$\frac{Ra}{l} + W_n \frac{a-k}{l} > W_1,\ W_2 \text{ will give the greatest shear} \qquad (1)$$

$$\frac{Ra}{l} + W_n \frac{a-k}{l} < W_1,\ W_1 \text{ will give the greatest shear} \qquad (2)$$

If, however, $k > a$ and

$$\frac{R}{l} > \frac{W_1}{a} \qquad W_2 \text{ will give the greatest shear}$$

$$\left.\begin{aligned}\frac{R}{l} &< \frac{W_1}{a}\\ \frac{R+W_n}{l} &> \frac{W_1}{a}\end{aligned}\right\} \text{ refer to equations (1) and (2).}$$

That is, the maximum shear at any point in the beam occurs when the foremost load is at that point, provided that the sum of all the loads on the beam, ΣW, is not greater than $\frac{W_1 l}{a}$; if ΣW is greater than $\frac{W_1 l}{a}$, the greatest shear occurs when some succeeding load is at the point.

ABSOLUTE MAXIMUM SHEARING FORCES :

The absolute maximum shear produced in a beam by a series of moving wheel loads will always occur at the end of the span, and usually just when the first (or second) of a series of heavy wheels is infinitely close to the end support, or on the point of moving off the beam.

In the example taken (Fig. 1) the controlling wheel is W_o, the first of the heavy wheels. At O, where the vertical through this load cuts the funicular polygon, the distance O H = Span A B is set off horizontally. At H drop a vertical to cut the link polygon in S. Join S to O. Through pole O draw a parallel to S O, cutting the load line in S_{AB}. Then $S_{AB}z$ is the absolute maximum shearing force for a span A B. Proceed similarly for other spans. Note that with the same loads, but slight differences from this case in their distances apart, the controlling wheel for shear might be W_9. This simplified form of graphical construction is amplified and extended by Mr. H. Bamford, M.Sc., Glasgow University.[1]

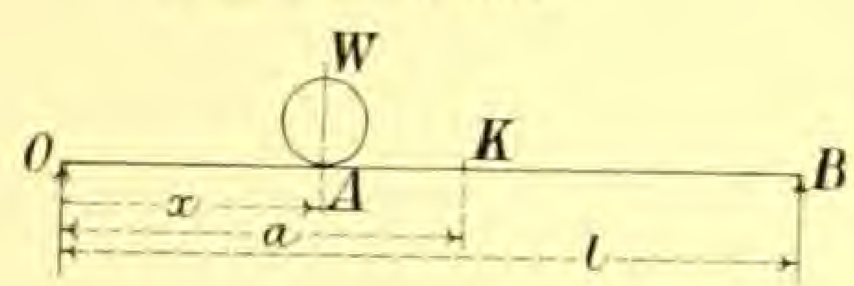
[1] See "Moving Loads on Railway Underbridges" (Whittaker, 1907).

Influence Lines.

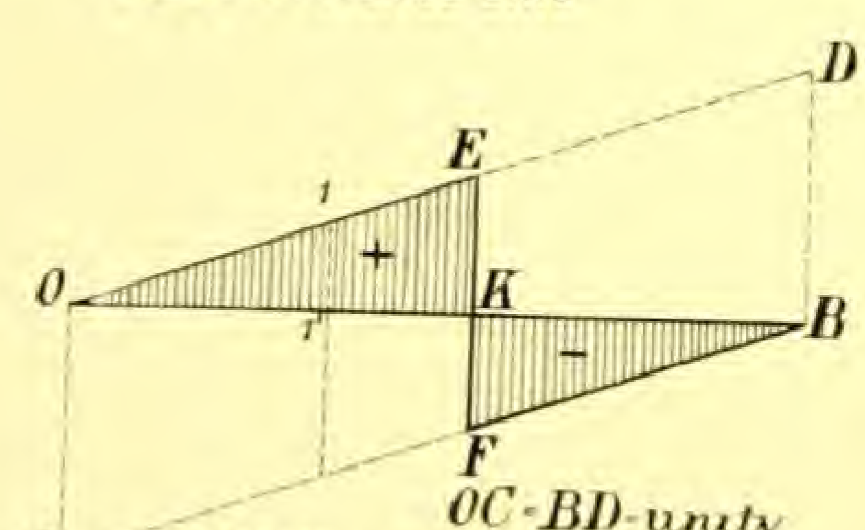
Definition.—An influence line is a curve representing the variation of any function, such as panel load, reaction, shearing force, bending moment, or stress at a particular section of a member due to unit load moving over the beam or structure. It represents the change in the function only for the section or point considered.

Case 1.—*Single Concentrated Load* W.

SYSTEM OF LOADING

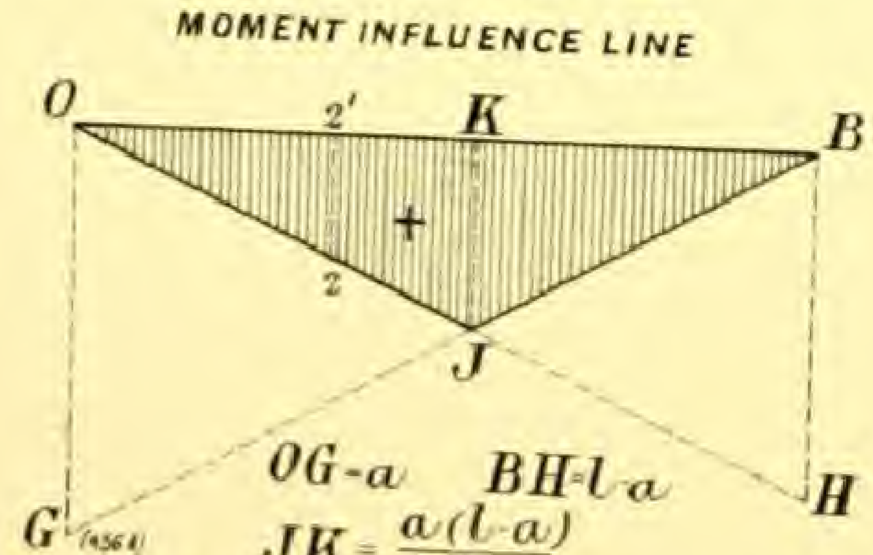


SHEAR INFLUENCE LINE



OEKFB is the influence line for shear at section K.

MOMENT INFLUENCE LINE



OJB is the influence line for moment at K.

Shear at Section K :

(i) $x < a$. $S_K = \frac{Wx}{l}$. For position of load shown $S_K = \frac{Wx}{l} = \frac{W \times O1'}{OB} = \frac{W \times 1'1}{BD} = W \times 1'1$ if BD be made unity.

(ii) $x > a$. $S_K = -\frac{W(l-x)}{l}$.

The shear at any section K for any position of the load W is the load W multiplied by the ordinate under W in that position.

Maximum $S_K = W \times KE = \frac{Wa}{l}$, or $-\frac{W(l-a)}{l}$, whichever is greater, and occurs when load is at K.

Bending Moment at K :

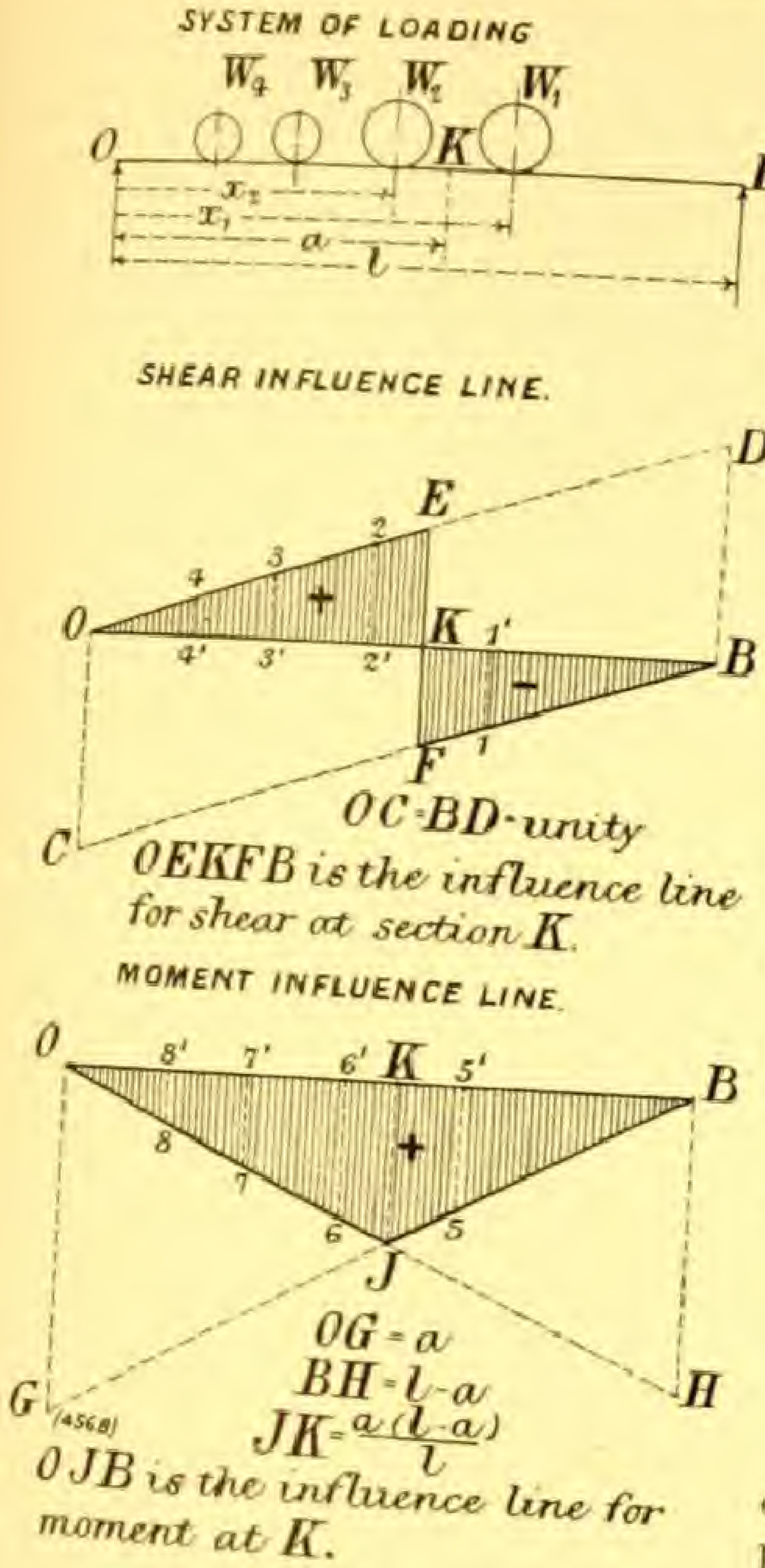
(i) $x < a$. $M_K = \frac{W(l-a)x}{l}$. For position of load shown $M_K = \frac{W(l-a)x}{l} = \frac{W \times BK \times O2'}{OB} = W \times BK \times \frac{2'2}{BH} = W \times 2'2 \times \frac{BK}{BH}$. It is convenient to make BK = BH. Then $M_K = W \times 2'2$.

(ii) $x > a$. $M_K = \frac{Wa(l-x)}{l}$.

Maximum $M_K = \frac{Wa(l-a)}{l} = W \times KJ$ when load is over section K.

The bending moment at any section K for any position of the load W is the product—W multiplied by ordinate under W in given position.

CASE 2.—*Series of Concentrated Loads*, W_1, W_2, W_3, *&c.*



SHEAR AT SECTION K:

(i) Loads to left of K. $S_K = \frac{1}{l} \Sigma Wx = \Sigma$ (W × ordinate under W).

(ii) Loads to right of K. $S_K = -\frac{1}{l} \Sigma W (l - x) = \Sigma$(W × ordinate under W.)

(iii) Loads on both sides of K. S_K = algebraic sum of (i) and (ii) above.

For position of loads shown. $S_K = -(W_1 \times 1'1) + (W_2 \times 2'2) + (W_3 \times 3'3) + (W_4 \times 4'4)$.

BENDING MOMENT AT K:

(i) Loads to left of K. $M_K = \frac{l - a}{l} \Sigma Wx$.

(ii) Loads to right of K. $M_K = \frac{a}{l} \Sigma W (l - x)$.

(iii) Loads on both sides of K. M_K = sum of (i) and (ii) above.

For position of loads shown. $M_K = (W_1 \times 5'5) + (W_2 \times 6'6) + (W_3 \times 7'7) + (W_4 \times 8'8)$.

GENERALLY:

The shear at any section K due to a series of moving concentrated loads in a given position is the algebraic sum of the products—load × ordinate to shear influence line, under load in given position.

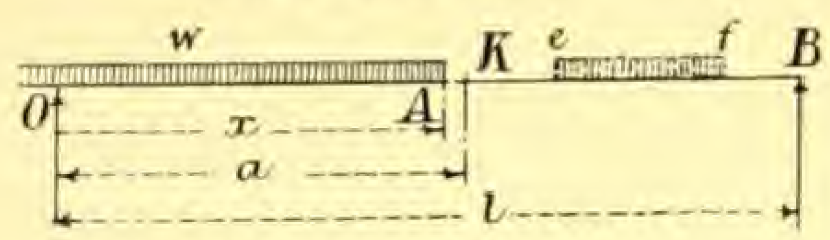
The bending moment at any section K due to a series of moving concentrated loads in a given position is the sum of the products—load × ordinate to moment influence line, under load in given position.

The maximum bending moment at a section K will occur when the heaviest loads are over or near that section, and *the maximum bending moment under any load occurs when the centre of the beam lies midway between that load and the centre of gravity of the series of loads.* This is found by trial. (See Section on Moving Loads, page 316.)

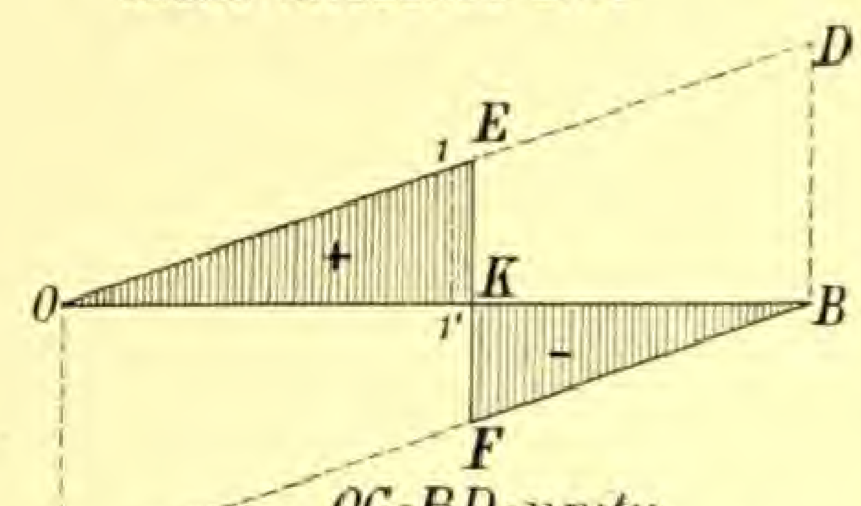
The maximum shear will occur at an abutment, and in general when the first or second of the heaviest leading wheels is at the abutment. (See page 317.)

CASE 3.—*Uniformly Distributed Moving Load w per foot.*

SYSTEM OF LOADING

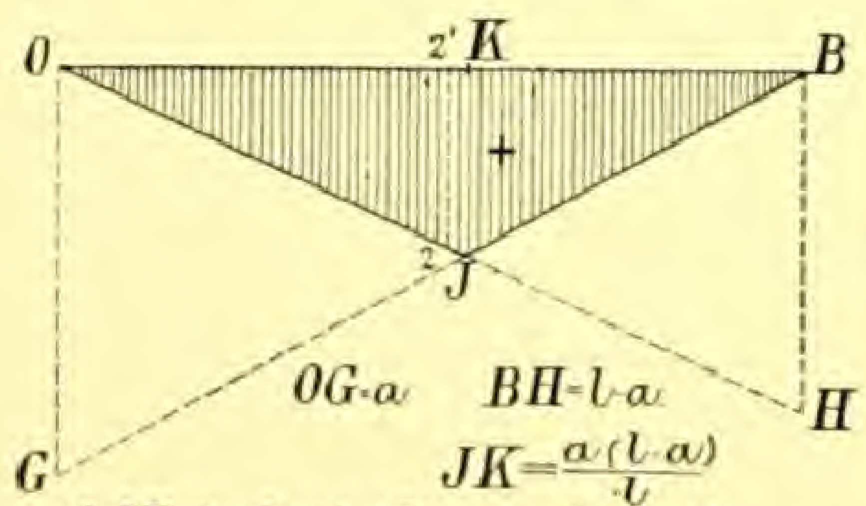


SHEAR INFLUENCE LINE



OEKFB is the influence line for shear at section K.

MOMENT INFLUENCE LINE.



OJB is the influence line for moment at K.

(3*a*) *Load of Indefinite Length.*

SHEAR AT SECTION K:

(i) $x < a.$ $\quad S_K = \dfrac{wx^2}{2l}.$

For position of load shown $S_K = \dfrac{wx^2}{2l} = \dfrac{w \times (O1')^2}{2\,OB}$

$= \dfrac{w \times O1'}{2} \times \dfrac{O1'}{OB}.$

But $\dfrac{O1'}{OB} = \dfrac{11'}{BD} = 11' \therefore S_K = \dfrac{w \times O1'}{2} \times 11' = w$ $\times$ area $O1'1$.

(ii) $x > a.$ $\quad S_K = \dfrac{w}{2l}\Big[x^2 - 2lx + 2al\Big].$

IN GENERAL:

The shear at any section K for any position of the load w (per foot run) is $w \times$ the area of the shear influence line diagram *up to that position.* If the front of the load be past the section the algebraic sum of the areas on the two sides of the section must be taken.

Maximum shear at K $= \dfrac{wa^2}{2l}$ or $w\dfrac{(l-a)^2}{2l}$ whichever is greater.

BENDING MOMENT AT SECTION K:

(i) $x < a.$ $\quad M_K = \dfrac{w(l-a)x^2}{2l}.$

For position of load shown $M_K = \dfrac{w(l-a)x^2}{2l} = \dfrac{w \times O2'}{2}$ $\times O2' \times \dfrac{KB}{OB}.$

But $O2' \times \dfrac{KB}{OB} = O2' \times \dfrac{BH}{OB} = \dfrac{O2' \times 2'2}{O2'}$

$\therefore M_K = \dfrac{w \times O2' \times 2'2}{2} = w \times$ area $O2'2$.

(ii) $x > a.$ $\quad M_K = \dfrac{wa}{2l}\Big[2lx - al - x^2\Big].$

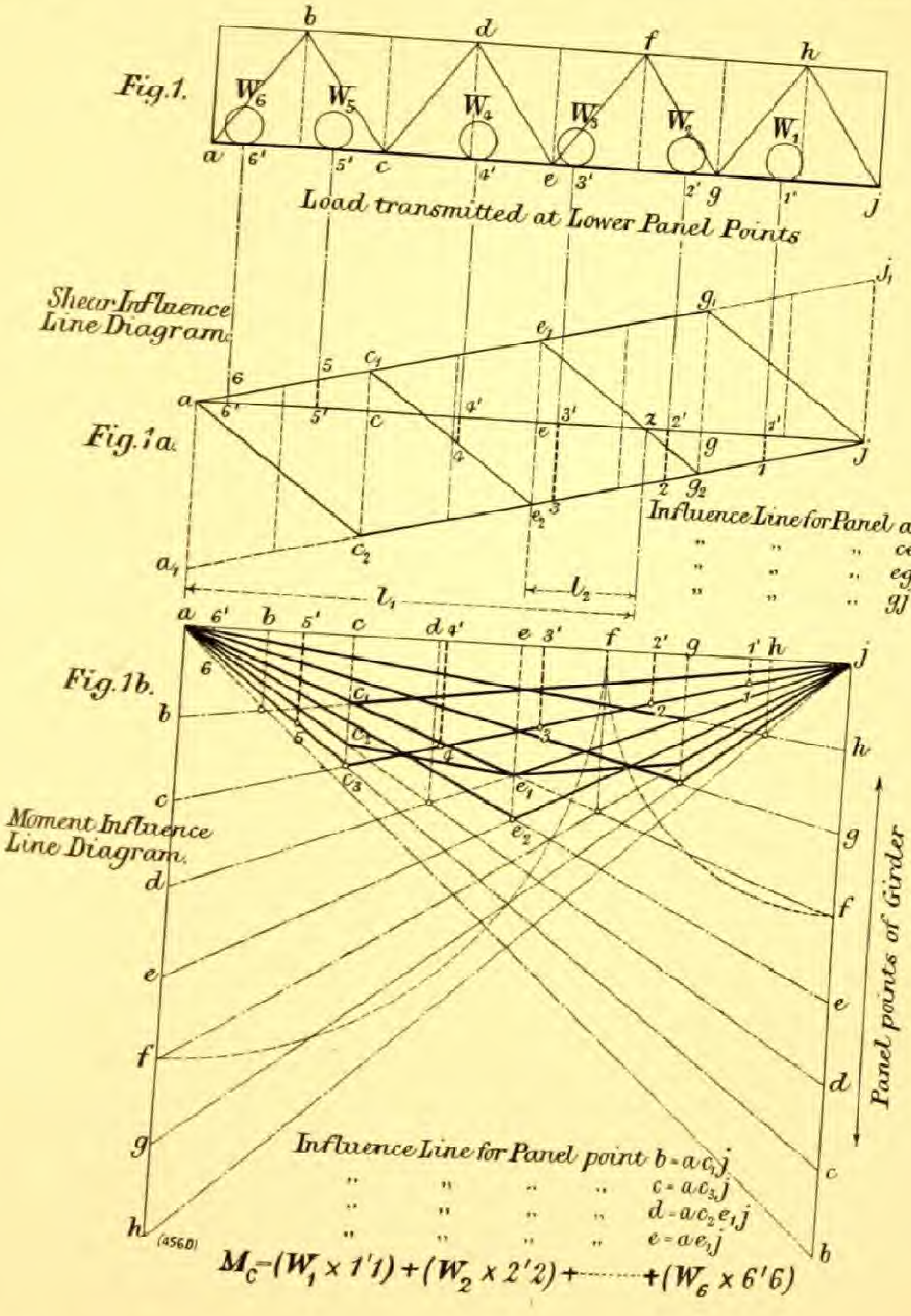
IN GENERAL:

The bending moment at any section K for any position of the load w (per foot run) is $w \times$ the area of the moment influence line diagram up to that position.

(3*b*) For a short distributed load like ef, take from the influence line diagram the area enclosed between the extreme ordinates vertically under e and f.

GIRDERS WITH PANELS.

The method of influence lines is specially applicable in the case of panelled trusses. An influence line is first drawn for the section at each panel point. It can be proved that the moment influence line between any two adjacent points at which the load is transmitted, for example between c and e, Fig. 1, must be a straight line. This explains the form of such a line as ac_2e_1j for panel point d, Fig. 1b.



Bending Moment.—The bending moment at any section for any position of the loads is computed by adding the products (ordinate under load to influence line for given section × load). Where the wheel loads are all the same or a definite ratio exists between them, this process is much simplified.

Maximum Bending Moment.—The criterion is: the average unit load on the left of the section must be equal to the average unit load on the whole span. The unit of length may be taken as a foot, but it is usually convenient in trusses with equal panels to take it as a panel length. The condition may be expressed thus. Consider a section at c, Fig. 1. Let W_{ac} = sum of loads between a and c; W_{aj} = sum of loads between a and j. Then for maximum bending moment at c $\frac{W_{ac}}{ac} = \frac{W_{aj}}{aj}$. For example let $W_1 = W_2 = W_3 = W_4 = 2\frac{1}{4}$ tons, $W_5 = W_6 = 1\frac{1}{2}$ tons. Then for maximum moment at c we have, trying the position of loads shown in Fig. 1, $\frac{W_{ac}}{ac} = 3$, $\frac{W_{aj}}{aj} = \frac{12}{4} = 3$, which satisfies the condition. There are generally two or more positions of a given load system which will satisfy the criterion. The wheels should be drawn on tracing paper and moved along the girder into positions satisfying the condition. It will usually be sufficient to compute two or perhaps three separately. The greatest is taken. The criterion also gives minimum values if it is satisfied when a load is at a or j, but a maximum value requires a load to be placed at a point where the angle in the influence line is convex upwards, and a minimum when it is convex downwards.

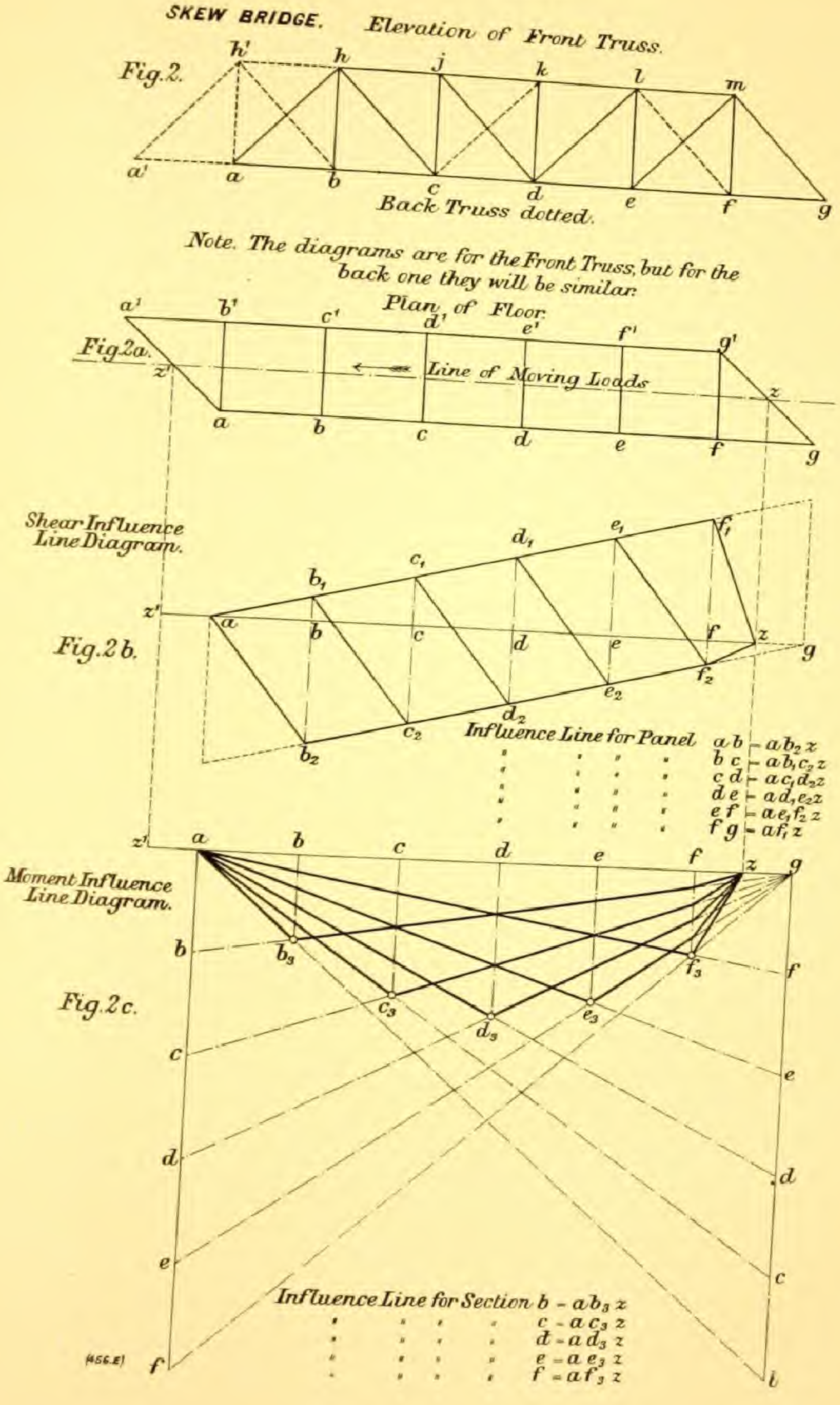
Shearing Force.—The shear in any panel for a given position of the loads is computed by adding the products (ordinate under load to shear influence line for given panel × load).

Maximum Shearing Force.—The criterion is: the sum of the loads in any panel must be equal to the average panel load on the girder. Let ΣW_p = loads in the panel, ΣW = the total load on the truss, n = the number of panels; then $\frac{\Sigma W}{n} = \Sigma W_p$. For any particular wheel W_r placed at the panel point to the right of a panel there will be a maximum shear in that panel so long as the sum of the loads on the girder ΣW lies between $n W_p$ and $n(\Sigma W_p + W_r)$. That is ΣW is greater than $n \Sigma W_p$ and less than $n(\Sigma W_p + W_r)$ where ΣW_p is the sum of the wheels in the panel other than the wheel W_r. The maximum shear in a panel will usually occur when the first or second wheel is near the beginning of that panel.

Maximum Web Stress.—In general no load must pass the panel in question. Consider web member fg, Fig. 1. Let $az = l_1$ be the distance from the left end to the point at which a load causes no stress in fg, and $ez = l_2$ the distance from the left end of the panel to the same point z. The condition then is $\frac{\Sigma W}{\Sigma W_p} = \frac{l_1}{l_2}$. For the greatest counter stress, l_1 becomes $(l - l_1)$ and l_2 becomes $(l - l_2)$. It is usually necessary to place a load at the panel point e and consider only as much of it as is necessary to satisfy the condition.

Skew Bridge.—The case of a bridge on the skew is treated in Figs. 2, 2a, 2b, and 2c. The amount of the skew is one panel length and the girder shown in full is the one treated. $z'z$ is the line of the moving loads and need not necessarily be along the centre of the bridge. The diagrams in Figs. 2b and 2c are similar to the last except at the right-hand end where the influence of the skew is felt.

Girders with Subdivided Panels.—These require special consideration only for the maximum web stresses, which, however, may be obtained by the methods already given. Fig. 3 shows a panel of a representative type. It should be noticed that main members will have their maximum stress when the loading covers the longer portion of the girder and counter members when it covers the shorter portion. In the case shown for loading at the bottom panel points there will be no stress in G H or E H. The bracing may act either of the ways indicated by dotted lines.


SKEW BRIDGE. Elevation of Front Truss.
Fig. 2.
Back Truss dotted.
Note. The diagrams are for the Front Truss, but for the back one they will be similar.
Plan of Floor.
Fig. 2a.
Line of Moving Loads
Shear Influence Line Diagram.
Fig. 2 b.
Influence Line for Panel ab - ab_2z
" " " " bc - ab_1c_2z
" " " " cd - ac_1d_2z
" " " " de - ad_1e_2z
" " " " ef - ae_1f_2z
" " " " fg - af_1z
Moment Influence Line Diagram.
Fig. 2 c.
Influence Line for Section b - ab_3z
" " " " c - ac_3z
" " " " d - ad_3z
" " " " e - ae_3z
" " " " f - af_3z
(456.E)

For panel D B the criterion given above may be applied. In Fig. 4 we must subtract the compression due to D H as part of D A and a member of the trussed stringer, from the tension in D H found by the application of the criterion, and in Fig. 5 the stress in A H must be

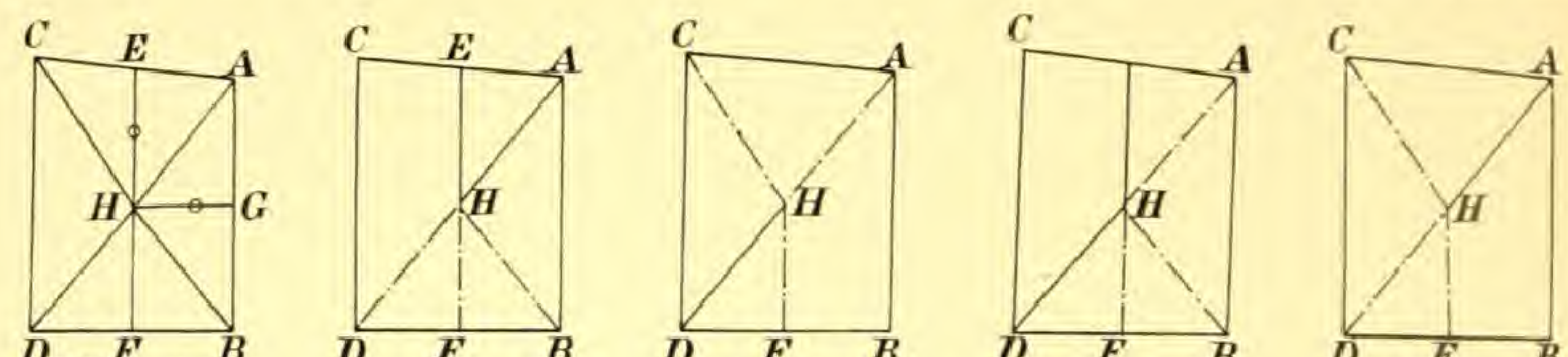


Fig. 3. Fig. 4. Fig. 5. Fig. 6. Fig. 7.

increased similarly. We may apply the criterion at once to D H in Fig. 6. The panel length is here D F. A H in Fig. 7 is treated in the same way. The greatest stress in the rest of the member A D in Figs. 6 and 7 is then taken to be the same as that already found for the other part, but increased or decreased by the stress in the portion dotted due to it being a part of the subordinate bracing. This, however, does not give the greatest maximum stress that may occur in A H, Fig. 6, or in D H, Fig. 7, but the error involved will be small.

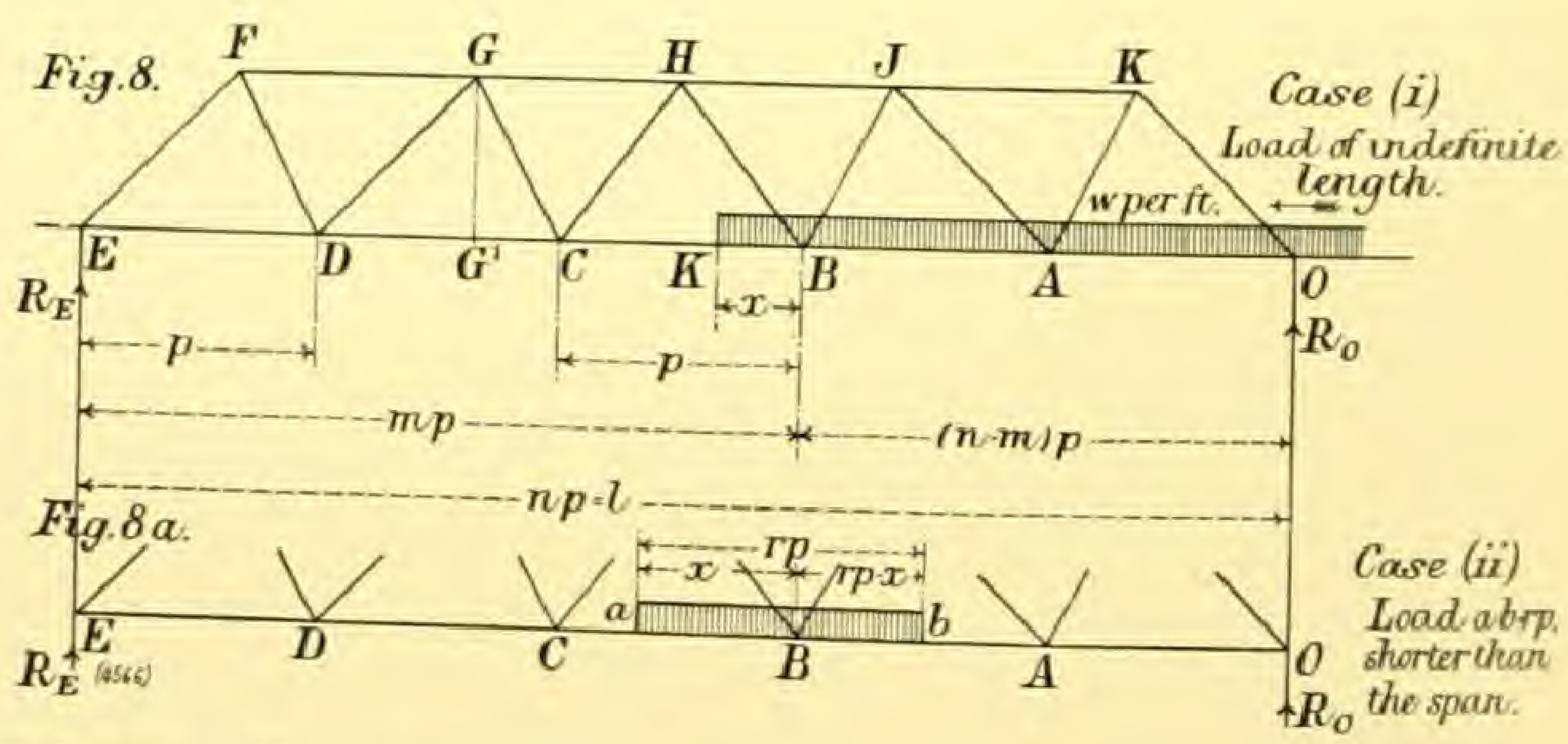
UNIFORMLY DISTRIBUTED MOVING LOADS ON PANELLED GIRDERS.

Bending Moment.—The influence line for any section will be similar to those in Fig. 1*b*.

Case (i).—Let M_c = moment at any panel point C, due to a uniform load of w per unit length moving on the girder from the right. Take the influence line for a section at panel point C. Let Δ be the area of this influence line diagram up to an ordinate vertically under the front of the load. Then $M_c = w \times \Delta$.

Case (ii).—Δ is now the area between the ordinates at a and b vertically under the front and back of the load. $M_c = w \times \Delta$.

Fig. 8.



Maximum Bending Moment:

Case (i).—The maximum bending moment for any section occurs when the load covers the whole span, and is equal to $\frac{w}{2} d (l - d)$ where d is the distance of the panel point considered from the left end.

As the influence line would show, this equation does not hold for a panel point on the top boom not vertically above one on the lower boom, as F, G, H, etc., Fig. 8. But the moment at any point, *e.g.*, G, is found easily from the diagram or by getting the moments at D and C and interpolating, since the moment varies as a straight line between D and C. Thus

$$M_G = M_D + \frac{DG'(M_C - M_D)}{DC}.$$

Case (ii).—The maximum at any panel point will occur for some position of the load over the panel point. Referring to Fig. 8*a*, bending moment at

$$B = M_B = (R_E \times mp) - \frac{wx^2}{2} = \frac{wrmp}{2n}\left\{p(n-m) + x - \frac{rp}{2}\right\}^2 - \frac{wx^2}{2}.$$

This is a maximum for $x = \dfrac{rmp^2\{r - 2(n-m)\}}{2(rmp - n)}$.

SHEARING FORCE.—The influence line for any section will be similar to those in Fig. 1*a*.

Case (i).—Let S_{BC} be the shear in panel BC when front of load is at any position K. Take the influence line for panel BC. Let Δ be the area of this diagram *taken algebraically* up to an ordinate vertically under K. Then $S_{BC} = w \times \Delta$.

Case (ii).—Δ is now the area between the extreme ordinates at *a* and *b*.

MAXIMUM SHEAR:

Case (i).—(*a*) True value. The maximum shear in any panel, say BC, will occur when the load w per unit length extends some distance into the panel. Let x be that distance. The shear is then $R_E - W_C$ where W_C is the load transferred to panel point C. Let p be the panel length, and letter the girder as shown in Fig. 8. Then by moments about B we obtain $W_C = \dfrac{wx^2}{2p}$, and by moments about O, $R_E = \dfrac{w}{2l}\left\{p(n-m) + x\right\}^2$. Therefore, shear in panel BC $= S_{BC} = R_E - W_C = \dfrac{w}{2l}\left\{p(n-m) + x\right\}^2 - \dfrac{wx^2}{2p}$. The maximum value of this shear, S_{BC} max., will occur when $x = \dfrac{p(n-m)}{n-1}$. Then S_{BC} max. $= \dfrac{wp}{2} \cdot \dfrac{(n-m)^2}{n-1}$.

Case (i).—(*b*) Approximate conventional value. The panel points on one side of the panel considered are assumed fully loaded, and those on the other side unloaded. This is an impossible condition. By moments about O, $R_E = S_{BC} = \dfrac{wp}{2n}(n-m)(n-m+1)$. This gives a slightly larger value of the shear than the true equation, and the difference is greatest when $m = \dfrac{n+1}{2}$, that is, at the centre of span, where it is approximately equal to $\dfrac{wp}{8}$ for any span.

Case (ii).—The true maximum shear in any panel will usually occur when the end of the load projects over into a panel. Referring to Fig. 8*a*, left hand reaction $R_E = \dfrac{wr}{2n}\left\{p(n-m) + x - \dfrac{rp}{2}\right\}^2$. Load at panel point $C = W_C = \dfrac{wx^2}{2p}$. Shear in panel CB $= S_{BC} = \dfrac{wr}{2n}\left\{p(n-m) + x - \dfrac{rp}{2}\right\}^2 - \dfrac{wx^2}{2p}$. The maximum shear will occur for $x = \dfrac{rp^2\{r - 2(n-m)\}}{2(rp - n)}$.

Columns with Eccentric Loads.

W = Central Load. A = Area of Column. r = Radius of Gyration. P = Eccentric Load

I = Moment of Inertia assumed constant. E = Modulus of Elasticity assumed uniform.

y = Distance of Extreme Fibre from axis of bending.

R_O, R_C, R_D = Reactions at O, C, D due to P. M_O, M_A, M_B = Bending Moment at O, A, B due to P.

f_1 = Maximum Unit Stress at side of column due to direct compression.

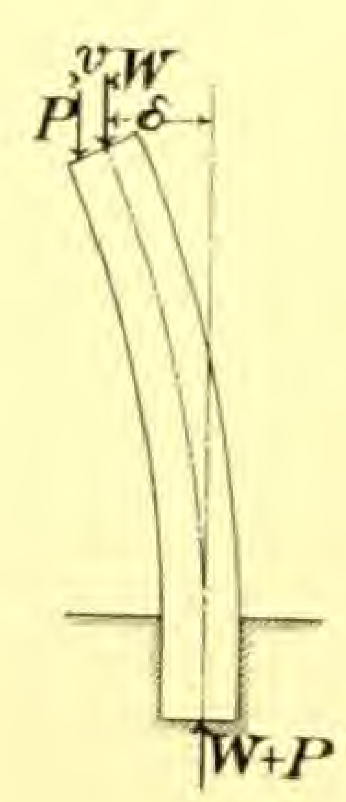
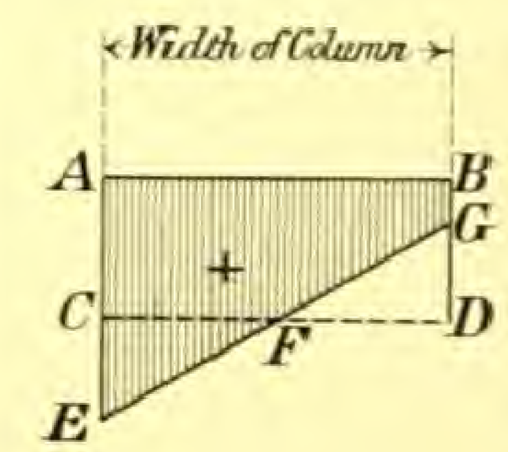
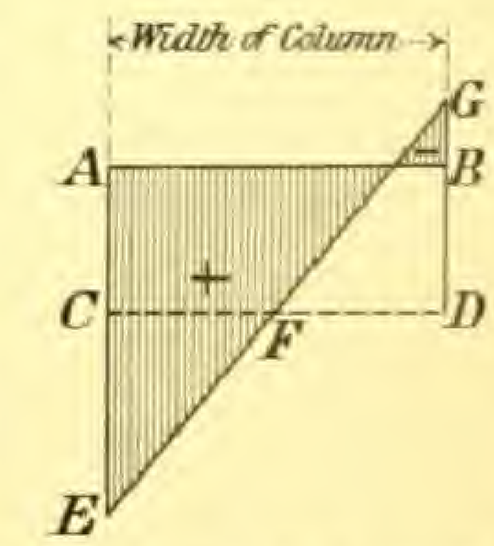
f_2 = Maximum Unit Stress at side of column due to bending.

$f = f_1 \pm f_2$ = Maximum Total Unit Stress at side of column due to both direct and bending stresses.

NOTE.—The Plus or Minus sign in the formula to be taken according as the stress due to bending is of the same kind as, or of opposite kind to, $\frac{W}{A}$.

---

(1) *Fixed at one end, free at the other. Eccentric Load within breadth of the column.*

	SYSTEM OF LOADING.	DISTRIBUTION OF STRESS ON CROSS SECTION (RECTANGULAR).	
		(i) No Tension. P within middle third of section.	*(ii) With Tension. P outside middle third of section.*
			
		Column not deflected. $\delta = 0$ $AC = BD = \frac{W+P}{A}$ $CE = DG = \frac{Pv}{z}$; $\left(z = \frac{I}{y}\right)$ or $f = \frac{1}{A}\left\{W + P\left(1 \pm \frac{6v}{AB}\right)\right\}$ *for rectangular section.*	*Column not deflected.* $\delta = 0$ $AC = BD = \frac{W+P}{A}$ $CE = DG = \frac{Pv}{z}$ or $f = \frac{1}{A}\left\{W + P\left(1 \pm \frac{6v}{AB}\right)\right\}$ *for rectangular section.*

Reaction at base of column = W + P

Max. B M = P $(\delta + v)$ + W δ

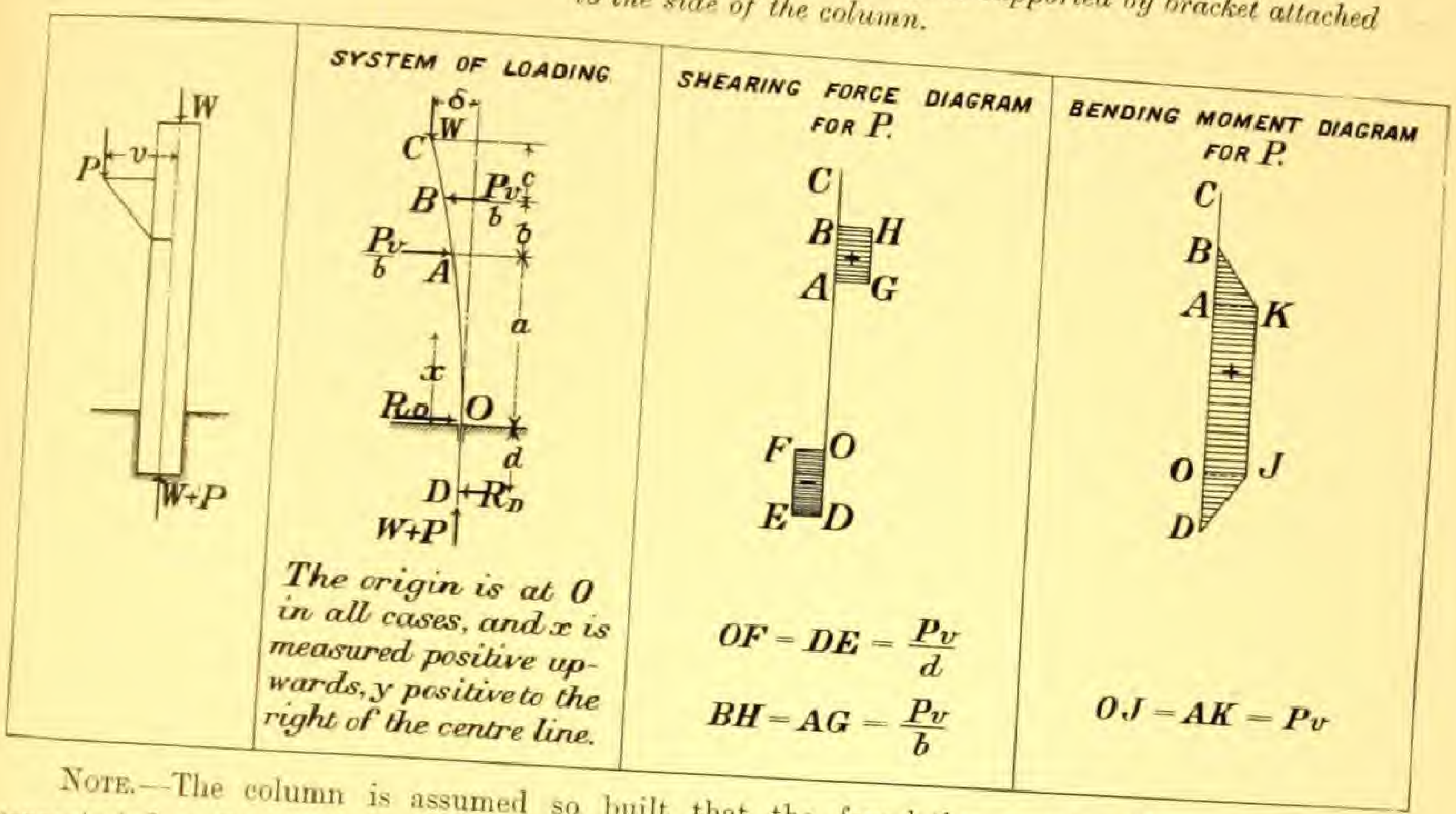
$$f_1 = \frac{W + P}{A}$$

$$f_2 = \frac{\{P(\delta + v) + W\delta\}\,y}{A\,r^2}$$

$$f = \frac{W + P}{A} \pm \frac{y}{A\,r^2}\left\{Pv + \delta(W + P)\right\} \text{ for deflected column}$$

$$f = \frac{W + P}{A} \pm \frac{Pv\,y}{A\,r^2}, \text{ when } \delta \text{ can be neglected.}$$

(2) *Fixed at one end and free at the other. Eccentric Load supported by bracket attached to the side of the column.*



NOTE.—The column is assumed so built that the foundation reaction R_D can be taken as concentrated at D.

$$R_o = R_D = \frac{Pv}{d}$$

Shear between A and B $= \dfrac{Pv}{b}$

Shear between O and D $= \dfrac{Pv}{d}$

$M_o = M_A =$ Maximum Bending Moment $= + Pv$

Max. deflection due to eccentric load:

$$\delta_c = -\frac{Pv}{6\,EI}\left[3\,a^2 + 6\,ab + 2\,b^2 + 3\,c\,(2\,a + b)\right]$$

$$f = \frac{W + P}{A} \pm \frac{y}{A\,r^2}\left\{Pv + \delta\,(W + P)\right\} \text{for deflected column}$$

$$f = \frac{W + P}{A} \pm \frac{Pvy}{A\,r^2}, \text{ when } \delta \text{ can be neglected.}$$

W = Central Load. A = Area of Column. r = Radius of Gyration. P = Eccentric Load.
I = Moment of Inertia assumed constant. E = Modulus of Elasticity assumed uniform.
y = Distance of Extreme Fibre from axis of bending.
R_O, R_C, R_D = Reactions at O, C, D due to P. M_O, M_A, M_B = Bending Moment at O, A, B due to P.
f_1 = Maximum Unit Stress at side of column due to direct compression.
f_2 = Maximum Unit Stress at side of column due to bending.
$f = f_1 \pm f_2$ = Maximum Total Unit Stress at side of column due to both direct and bending stresses.

NOTE.—The Plus or Minus sign in the formula to be taken according as the stress due to bending is of the same kind as, or of opposite kind to, $\frac{W}{A}$.

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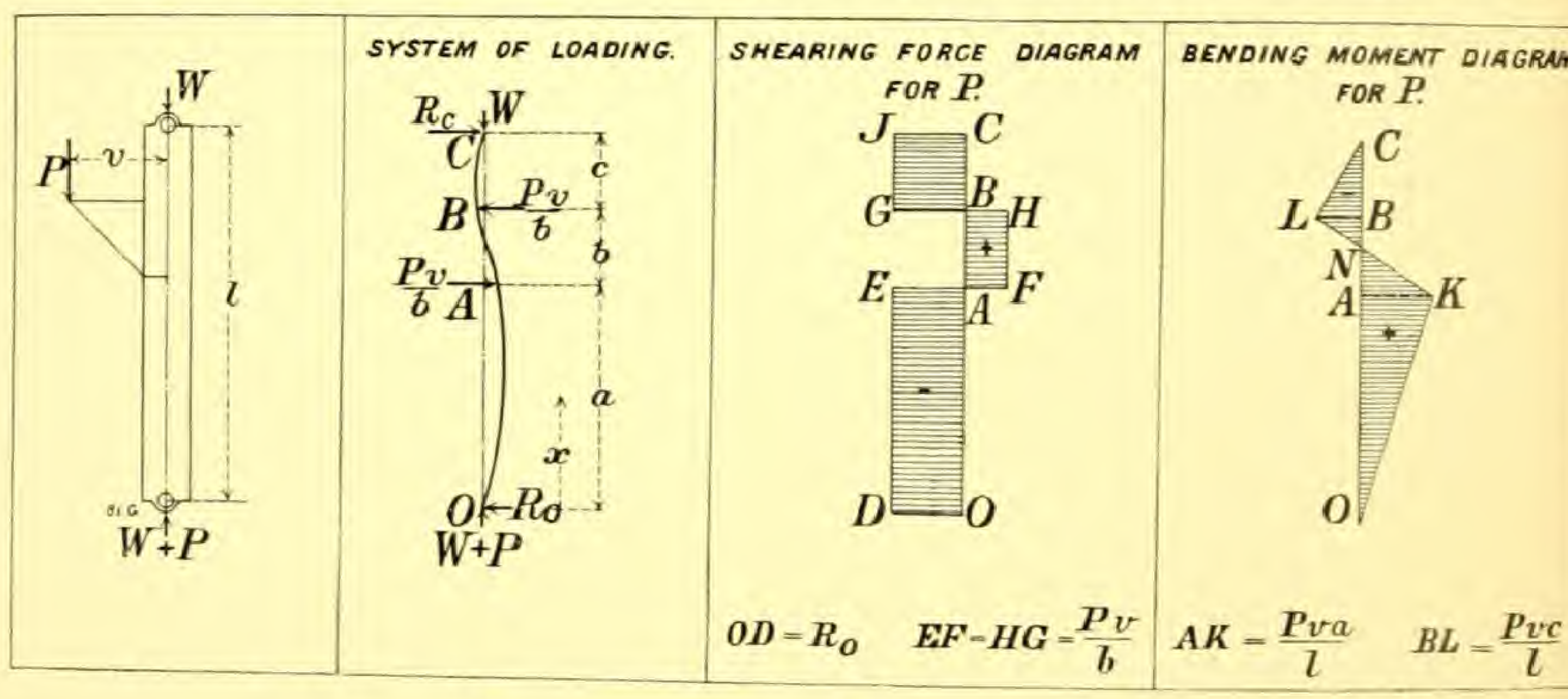
(3) *Pin-jointed at both ends. Eccentric Load supported by brackets attached to the side of the column.*



$$R_O = R_C = \frac{Pv}{l} \qquad M_A = \frac{Pva}{l} \qquad M_B = -\frac{Pvc}{l}$$

Maximum deflection: $x < a$.

$$\delta_{max.} = \frac{Pv\sqrt{m}}{9\sqrt{3}\,EI\,l} \text{ at point where } x = \sqrt{\frac{m}{3}}$$

$$\left[m = 6a^2 - b^2 + 3l(2a + b) - 2l^2\right]$$

Points of inflection:

$x > a$, and $< a + b$

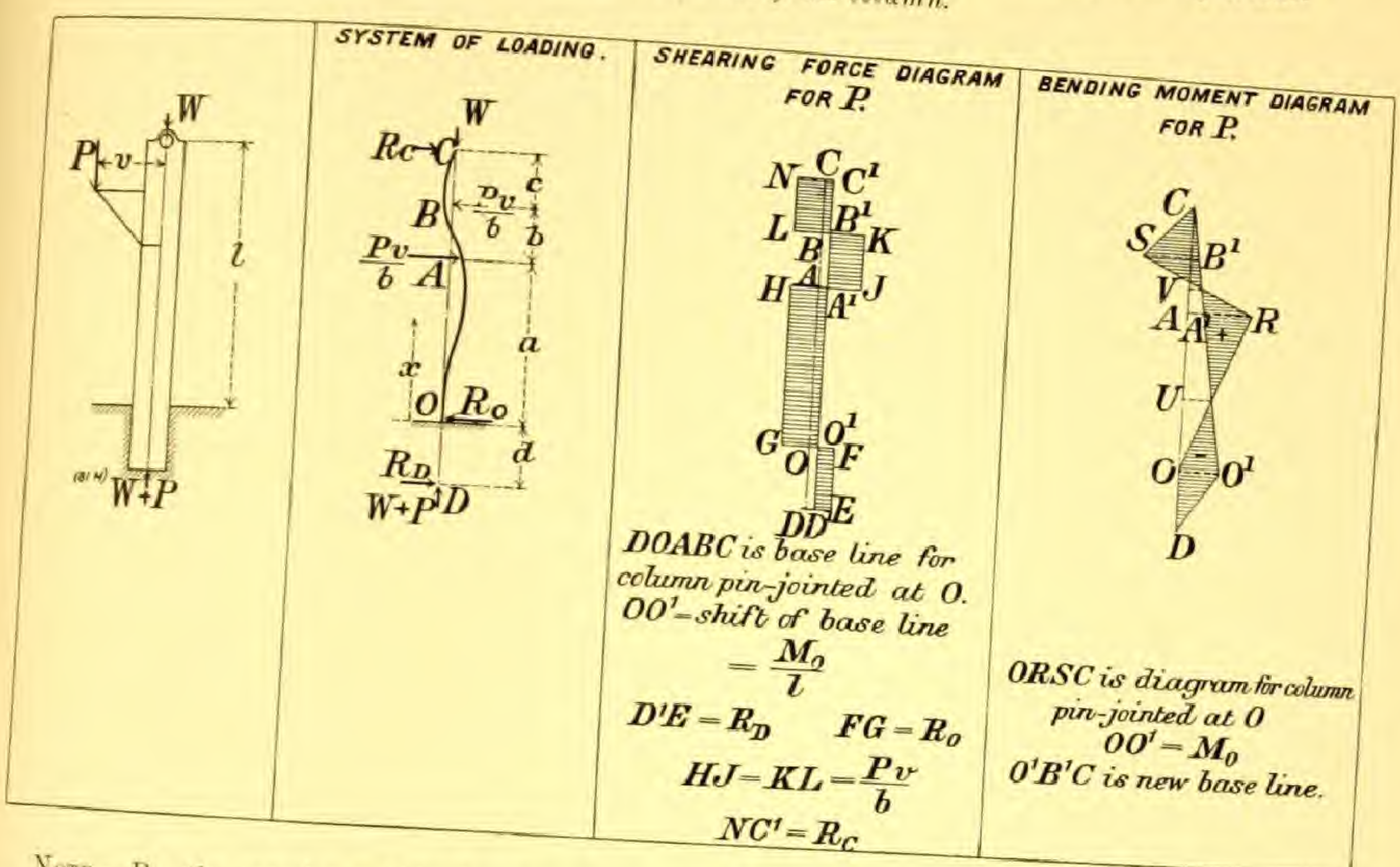
$$x = \frac{al}{l - b}$$

$$f_1 = \frac{W + P}{A}; \; f_2 = \left\{\frac{Pva + Wl\delta_A}{Ar^2 l}\right\} y$$

$$f = \frac{W + P}{A} \pm \frac{y}{Ar^2}\left(M_{max.} + W\delta_A\right). \text{ If } \delta = o, f = \frac{W + P}{A} \pm \frac{M_{max.}\,y}{Ar^2}.$$

δ is the deflection at point of maximum bending moment.

(4) *Pin-jointed at the top, and fixed at the bottom. Eccentric Load supported by bracket attached to the side of the column.*



NOTE.—R_D, the reaction within foundation of column, is assumed concentrated at D.

$$R_0 = +\frac{Pv}{2l^3}\left[3a^2 + 6ab + 2b^2 + 3c(2a+b)\right]$$

$$R_D = -\frac{R_0 l - Pv}{(l+d)} = -\frac{M_0}{d} \quad . \quad R_C = R_0 - R_D$$

$$M_0 = Pv - R_C l$$

$$M_A = Pv - R_0(l-a)$$

$$M_B = -R_C C$$

Maximum deflection: $x < a$

$$\delta_{max.} = -\frac{2M_0^3}{3R_0^2}; \text{ at point where } x = -\frac{2M_0}{R_0}$$

Points of inflection:

(i) $x < a$; $x = -\dfrac{M_0}{R_0} = OU$

(ii) $x > a$ and $< a + b$, $x = \dfrac{M_0 b + Pva}{Pv - R_0 b} = OV$

$f = \dfrac{W+P}{A} \pm \dfrac{M_{max.} y}{Ar^2}$; at section of Max. Bending Moment.

Every expression here assumes E uniform and I constant.

NOTE.—For the case of any column with base at O, and so disposed that its centre line remains vertical at O, the same expressions will give the values required except in case of R_C, which is $= -R_0$

S.F. diagram:

$O'N = C'K = R_0 = R_C$

The part $O'FED'$ disappears.

B.M. diagram:

The part $OO'D$ disappears.

W = Central Load. A = Area of Column. r = Radius of Gyration. P = Eccentric Load.
I = Moment of Inertia assumed constant. E = Modulus of Elasticity assumed uniform.
y = Distance of Extreme Fibre from axis of bending.
R_O, R_C, R_D = Reactions at O, C, D due to P. M_O, M_A, M_B = Bending Moment at O, A, B due to P.
f_1 = Maximum Unit Stress at side of column due to direct compression.
f_2 = Maximum Unit Stress at side of column due to bending.
$f = f_1 \pm f_2$ = Maximum Total Unit Stress at side of column due to both direct and bending stresses.

NOTE.—The Plus or Minus sign in the formula to be taken according as the stress due to bending is of the same kind as, or of opposite kind to, $\frac{W}{A}$.

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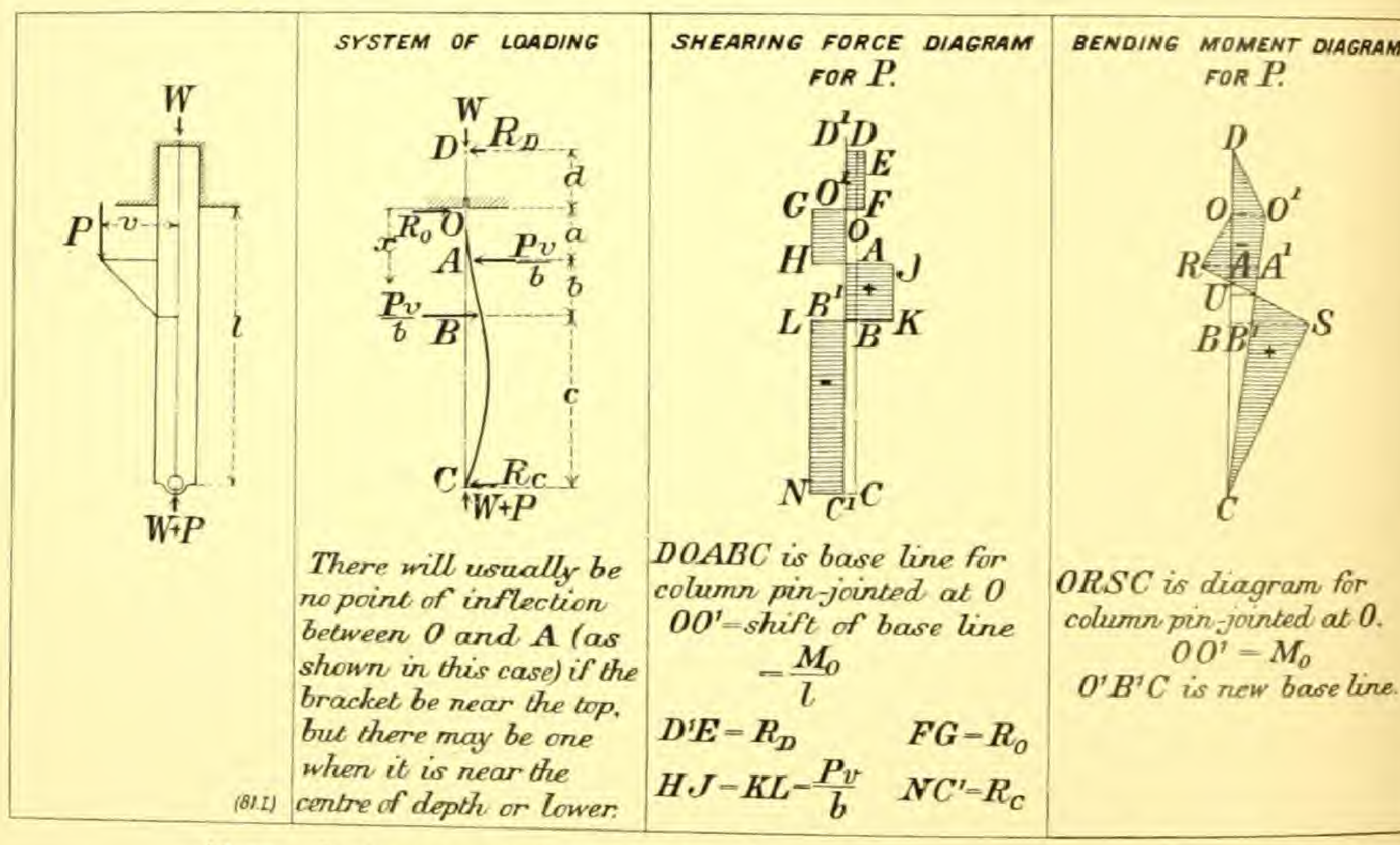
(5) *Pin-jointed at the bottom, and fixed at the top. Eccentric Load supported by bracket attached to the side of the column.*



NOTE.—R_D is assumed concentrated at D.

$$R_O = -\frac{Pv}{2\,l^3}\left[3\,a^2 + 6\,ab + 2\,b^2 + 3\,c\,(2\,a + b)\right]$$

$$R_D = \frac{M_O}{d} \qquad R_C = R_O - R_D \qquad M_O = -(Pv - R_C\,l)$$

$$M_A = -\{Pv - R_O\,(l - a)\} \qquad M_B = R_C\,c$$

Maximum deflection: $x < a$

$$\delta_{max.} = \frac{2\,M_O^3}{3\,R_O^2} \text{ at point where } x = \frac{2\,M_O}{R_O}$$

Points of inflection:

(i) $x < a$; $\quad x = \frac{M_O}{R_O}$

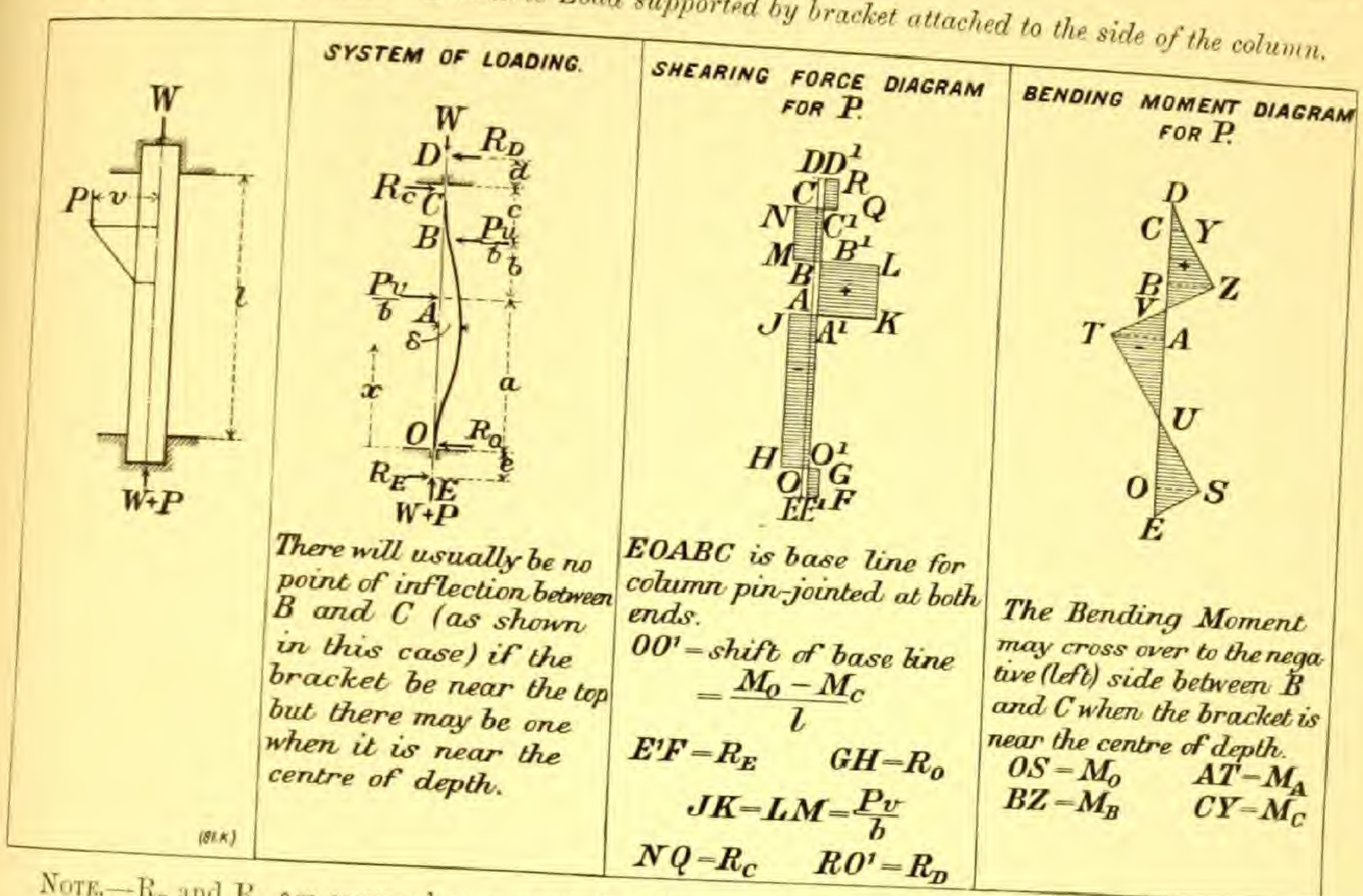
(ii) $x > a$ and $< (a + b)$ $\quad x = -\frac{M_O\,b + Pva}{Pv - R_O\,b} = OU$

$$f = \frac{W + P}{A} \pm \frac{M_{max.}\,y}{Ar^2} \text{ at section of Max. Bending Moment.}$$

NOTE.—For case of a column not continued above O, and so disposed that its centre line remains vertical at O, note restrictions similar to Case (4).

Every expression here assumes E uniform and I constant.

(6) *Fixed at both ends. Eccentric Load supported by bracket attached to the side of the column.*



NOTE.—R_E and R_D are assumed concentrated at E and D.

$$R_o = -\frac{Pv}{l^3}\left[b^2 + 3a(b+2c) + 3bc\right] \quad . \quad R_E = \frac{M_o}{l}$$

$$R_c = \frac{Pv}{l^3}\left[b^2 + 3c(b+2a) + 3ab\right] \quad . \quad R_D = \frac{M_c}{d}$$

$$M_o = \frac{Pv}{l^2}\left[-c^2 + ab + 2ac\right]$$

$$M_c = \frac{Pv}{l^2}\left[a^2 - bc - 2ac\right]$$

$$M_A = \frac{Pv}{l^3}\left[-2a^2(b+2c) + c^2(a-b) - c^3\right]$$

$$M_B = \frac{Pv}{l^3}\left[2c^2(b+2a) - a^2(c-b) + a^3\right]$$

Maximum deflection : $x < a$

$$\delta_{max.} = -\frac{2}{3}\frac{M_o^3}{R_o^2} \text{ at point where } x = -\frac{2M_o}{R_o}$$

Points of inflection :

(i) $x < a,\ x = -\frac{M_o}{R_o} = OU$

(ii) $x > a$, and $< (a+b),\ x = \frac{Pva + M_o b}{Pv - R_o b} = OV$

(iii) $x > (a+b) \quad . \quad x = \frac{M_C + R_C l}{R_C}$

$$f = \frac{W+P}{A} \pm \frac{M_{max.}\,y}{Ar^2}, \text{ at Section of Max. Bending Moment.}$$

NOTE.—For case of a column not continued beyond O and C, and so disposed as to remain vertical at O and C, the same expressions will give the values required.

S.F. diagram:

$O^1H = C^1N = R_o = R_c$

The parts O^1GFE^1 and C^1QRO^1 disappear.

B.M. diagram:

The parts OSE and CYD disappear.

Every expression here assumes E uniform and I constant.

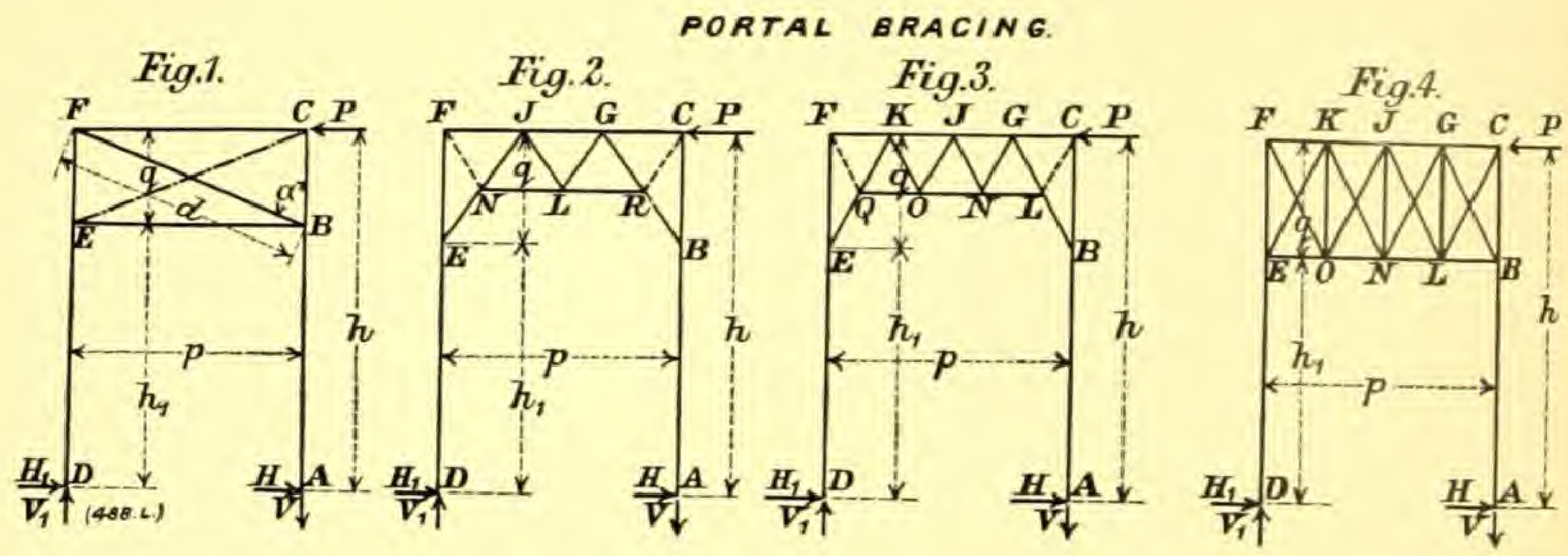
Portal Bracing.

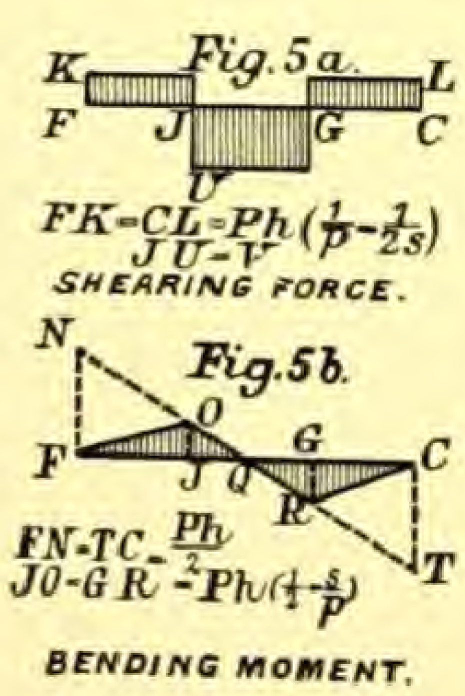
Signs :— + denotes compression, – denotes tension.

BENDING MOMENT :

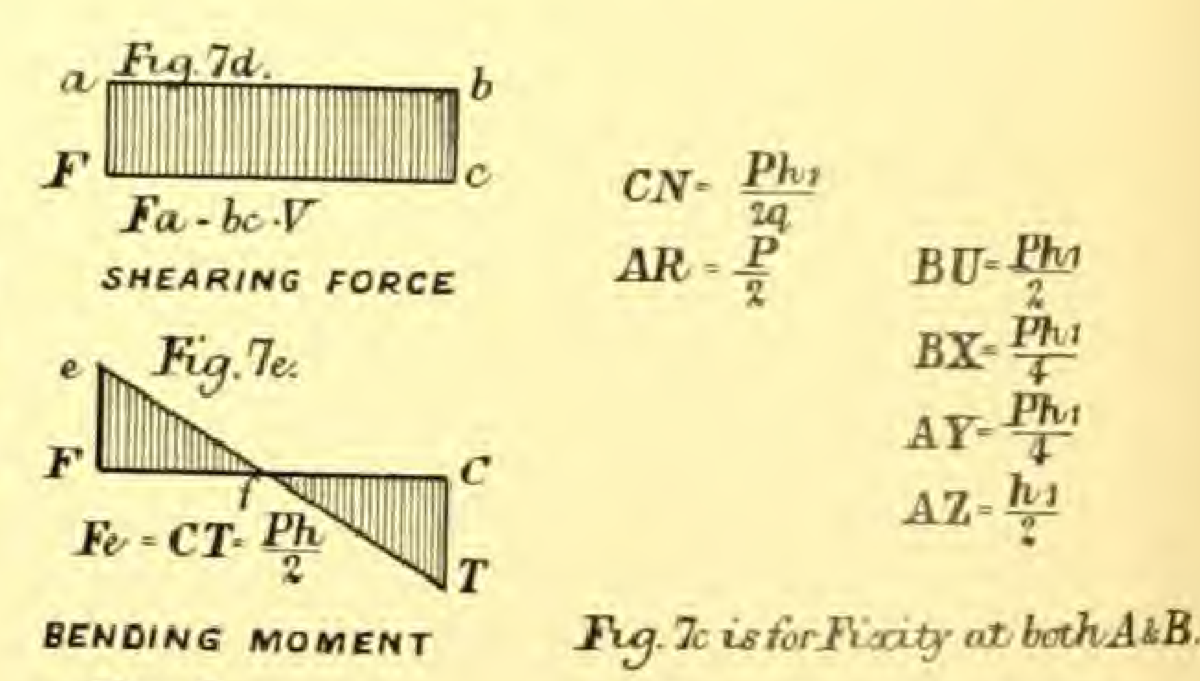
+ convex to left and plotted to left.

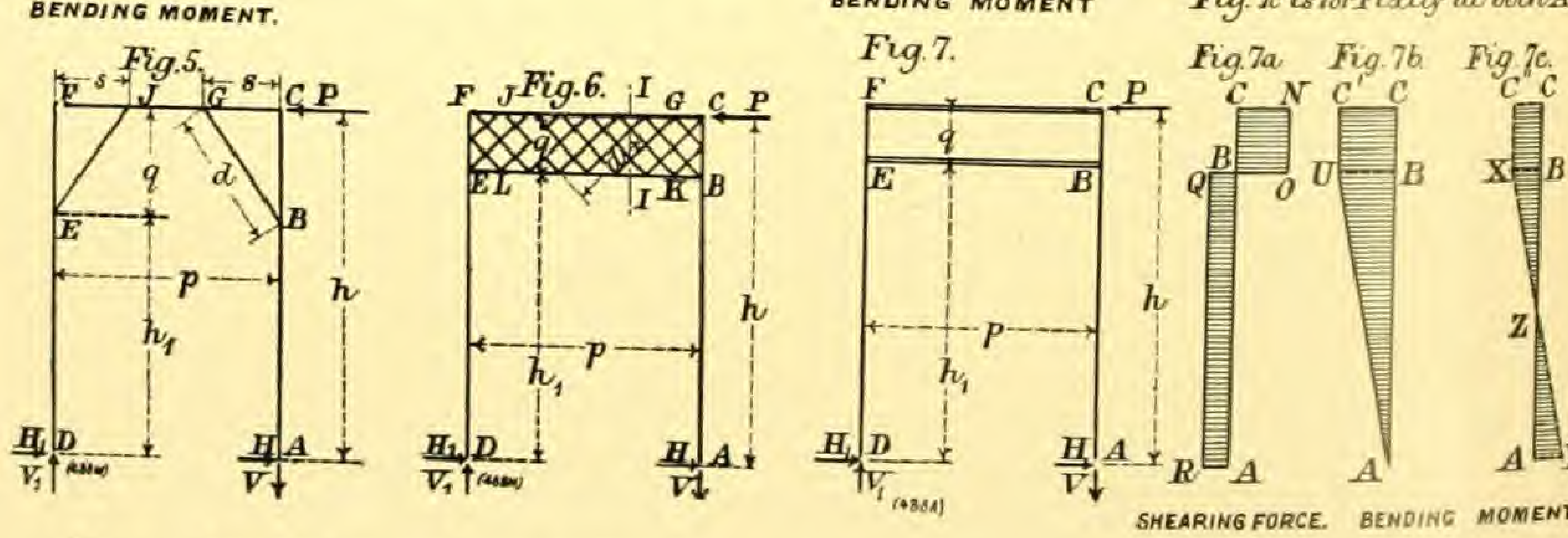
– convex to right and plotted to right.





The lattice portal (Fig. 6) is approaching the plate girder form (Fig. 7), which can usually be taken as fixed at the top end.





NOTE.—Any of the cases up to Fig. 11 may be treated as fixed at the base and top as well, by using the expressions under Leeward Column, V. page 338, but keeping H and H_1 equal to $\frac{P}{2}$.

I.—COLUMNS HINGED AT BASE.

FIGS. 1 to 7 represent some common types of portal bracing as used in buildings and bridge work. Other examples are given in Figs. 8, 9, 10, 11, and 12. The load brought to the portal (by the upper lateral members in the case of a bridge, for example) is considered as all applied at C, including the panel wind load acting at C and F. The sum of these loads is P. The reactions at A and D cannot be determined by ordinary statical equations. It is convenient and sufficiently accurate to take $H = H_1 = \frac{P}{2}$.

For cases, Figs. 1 to 4, diagrams of bending moment and shearing force in the posts are similar to those in Figs. 8A and 8B. They would be similar to Figs. 9A and 9B if the bases of the columns were fixed.

$$H = H_1 = \frac{P}{2}$$

$$V_1 = -V = \frac{Ph}{p}.$$

MAXIMUM BENDING MOMENT:

$$M_{max.} \text{ in posts } = Hh_1 = \frac{P}{2} h_1 \text{ at B and E.}$$

SHEARING FORCE:

$$\text{Shear in posts below B or E} = H = \frac{P}{2}$$

$$\text{,, ,, above ,,} = \frac{Hh_1}{q} = \frac{Ph_1}{2q}.$$

STRESSES:

The maximum fibre stress occurs at E on the right side of the leeward post, where the stresses due to moment, V_1, and dead and live loads are all compressive.

Fig. 1 is a portal with simple diagonal bracing: if the diagonals are capable of resisting compressive stress CF and BE will be in tension. In the following results the diagonals are assumed to be incapable of resisting compression, and tie CE is consequently assumed inoperative.

The stresses in the diagonals and EB, will be numerically the same for either tension or compression diagonals.

Member - -	CF	EB	FB	CB	Shear in post between B and C $= \frac{Ph_1}{2q}$
Stress - -	$\frac{Ph}{2q} + \frac{P}{2}$	$\frac{Ph}{2q}$	$P\frac{hd}{pq}$	$\frac{Ph_1}{2q}$	

The stress in FC with diagonals as compression members $= -\frac{Ph}{2q} + \frac{P}{2}$.

In Figs. 2 and 3 the stresses are easily found by the method of sections.

In Fig. 4 it is usual to assume either (*a*) that the stresses are all taken by that system of bracing in which the diagonal ties are in tension, or (*b*) that the portal can be treated as two simply-braced portals, and calculate the stresses with a load $\frac{P}{2}$. Then add the results algebraically. Method (*a*) is simpler. For maximum moment, shear and stresses in column, see formulæ under Fig. 8.

Fig. 5.—This form is used where there is a lack of head-room. The portal strut FC is designed as a girder to take the maximum moment, shear and direct stress. For maximum moment, shear and stresses in columns, use formulæ already given.

Member	FJ	GC	JG	EF	BC	GB	JE
Stress -	$-\left(\frac{Ph}{2q}-\frac{P}{2}\right)$	$\frac{Ph}{2q}+\frac{P}{2}$	$\frac{P}{2}$	$\frac{Ph}{2s}-\frac{Ph}{p}$	$-\left(\frac{Ph}{2s}-\frac{Ph}{p}\right)$	$-\frac{Ph_1}{2q}\frac{d}{s}$	$+\frac{Ph_1}{2q}\frac{d}{s}$
Shear -	$Ph\left(\frac{1}{p}-\frac{1}{2s}\right)$	$Ph\left(\frac{1}{p}-\frac{1}{2s}\right)$	$V_1=\frac{Ph}{p}$	$\frac{Ph_1}{2q}$	$\frac{Ph_1}{2q}$	$+\frac{Ph}{2p}\frac{d}{q}$	$-\frac{Ph}{2p}\frac{d}{q}$

Stress in BC > + stress in DE if $s<\frac{p}{4}$.

The maximum bending moment on FC occurs at J and G and $= Hh - Vs = Ph\left(\frac{1}{2}-\frac{s}{p}\right)$.

The diagrams of bending moment and shearing force for the columns are similar to Figs. 8A and 8B.

Fig. 6.—The stresses in the bracing or web are found from the shear and in the flanges from the bending moment. The shear is constant, and for any section is equally distributed among the members cut. If n = number of systems of bracing cut by a vertical section as II, then vertical component of stress $S_V = \frac{V_1}{n} = \frac{Ph}{np}$. Half of the members are in tension and half in compression. The maximum compression in FC occurs at C, and is

$S_{GC} = +\frac{Ph}{2q}+\frac{P}{2}$. Also maximum tensile stress in FC $= S_{FJ} = -\frac{Hh_1}{q} = -\frac{Ph}{2q}+\frac{P}{2}$.

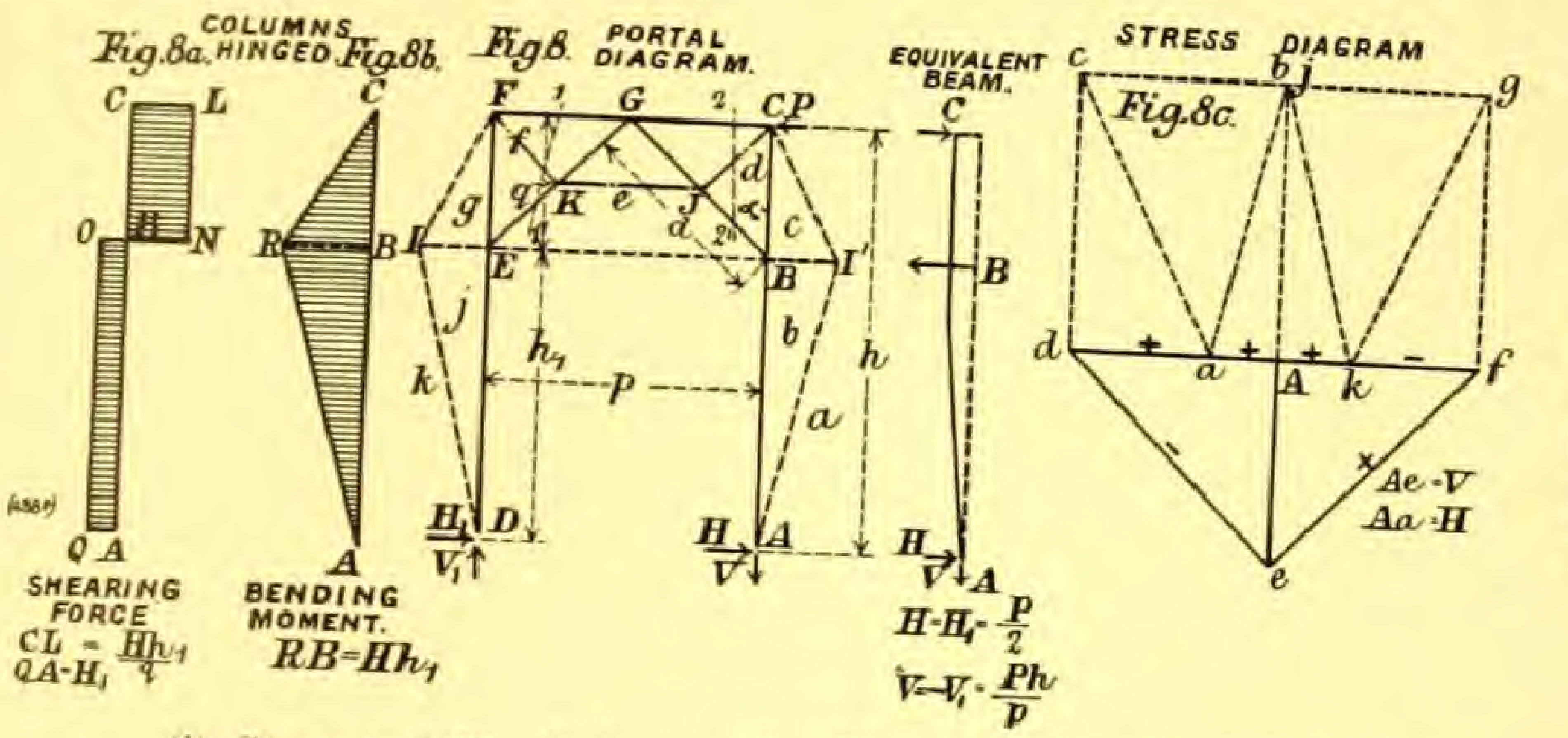
For lower flange maximum stress in EB, $S_{EB} = \pm\frac{Hh}{q} = \pm\frac{Ph}{2q}$. Maximum tension is at B and maximum compression at E.

Member -	FJ	GC	EL	KB	Diagonal
Stress -	$-\frac{Ph}{2q}\left(1-\frac{1}{n}\right)+\frac{P}{2}$	$+\frac{Ph}{2q}\left(1-\frac{1}{n}\right)+\frac{P}{2}$	$+\frac{Ph}{2q}\left(1-\frac{1}{n}\right)$	$-\frac{Ph}{2q}\left(1-\frac{1}{n}\right)$	$\pm\frac{Phd}{npq}$

Fig. 7 is a plate-girder portal. The shear at any point of the girder $= V_1 = \frac{Ph}{p}$. The columns ABC and DEF may be considered fixed along BC and EF.

Point	H	E	B	F	C	Centre
Flange stress	O	$+\frac{Ph}{2q}$	$-\frac{Ph}{2q}$	$-\frac{Ph}{2q}+\frac{P}{2}$	$+\frac{Ph}{2q}+\frac{P}{2}$	$+\frac{P}{2}$

In Fig. 8 the knee braces meet at the centre of the portal. This is a very common type, and will be taken as an example to show how the stresses may be found by the method of sections.



MEMBER	STRESS	SHEAR
AB	$\frac{Ph}{p}$	$\frac{P}{2}$
DE	$+\frac{Ph}{p}$	$\frac{P}{2}$
BC	0	$\frac{Ph_1}{2q}$
EF	0	Mmax in posts $\frac{Ph_1}{2}$ and occurs at B&E
GE	$\frac{Phd}{pq}$	
GB	$-\frac{Phd}{pq}$	
FG	$-\frac{Ph_1}{2q}$	
GC	$\frac{Ph}{2q}+\frac{P}{2}$	
JC	0	
FK	0	
KJ	0	

(i) Stress in GE. Take a section like 11[1] and take moments of the external forces to the left of the section about point F

$$S_{GE}\, q \sin \alpha = Hh \text{ or } S_{GE} = \frac{Hh}{q \sin \alpha} = \frac{Phd}{qp}.$$

(ii) Stress in GB. Take a section like 22[1] and take moments of the external forces to right of the section about C

$$S_{GB}\, q \sin \alpha = -Hh \text{ or } S_{GB} = -\frac{Phd}{qp}.$$

(iii) Stress in GC. Take a section like 11[1] and take moments about point B

$$S_{GC} \times q = Pq + Hh_1 \text{ or } S_{GC} = \frac{Pq + Hh_1}{q}.$$

(iv) Stress in FG similarly

$$S_{FG} \times q = -Hd \text{ or } S_{FG} = -\frac{Ph_1}{2q}.$$

The members shown chain-dotted receive no direct stress.

Graphical Solution.—There is a very simple and elegant graphical method of finding these stresses. It is due to Mr. Milo S. Ketchum, C.E.[1] Insert the framework FID and CI'A for the columns FD and CA respectively; H, H_1, V and V_1 remain as before and are known. Begin with a point A representing the foot of the column, and set off Aa = H and Ae = V as shown. Then ab and eb are stresses in corresponding members. The stresses in bc and ca are now easily found by closing the polygon, and others similarly, taking joints B, C and G in order, finishing up at D. The dotted lines in the stress diagram are only auxiliary.

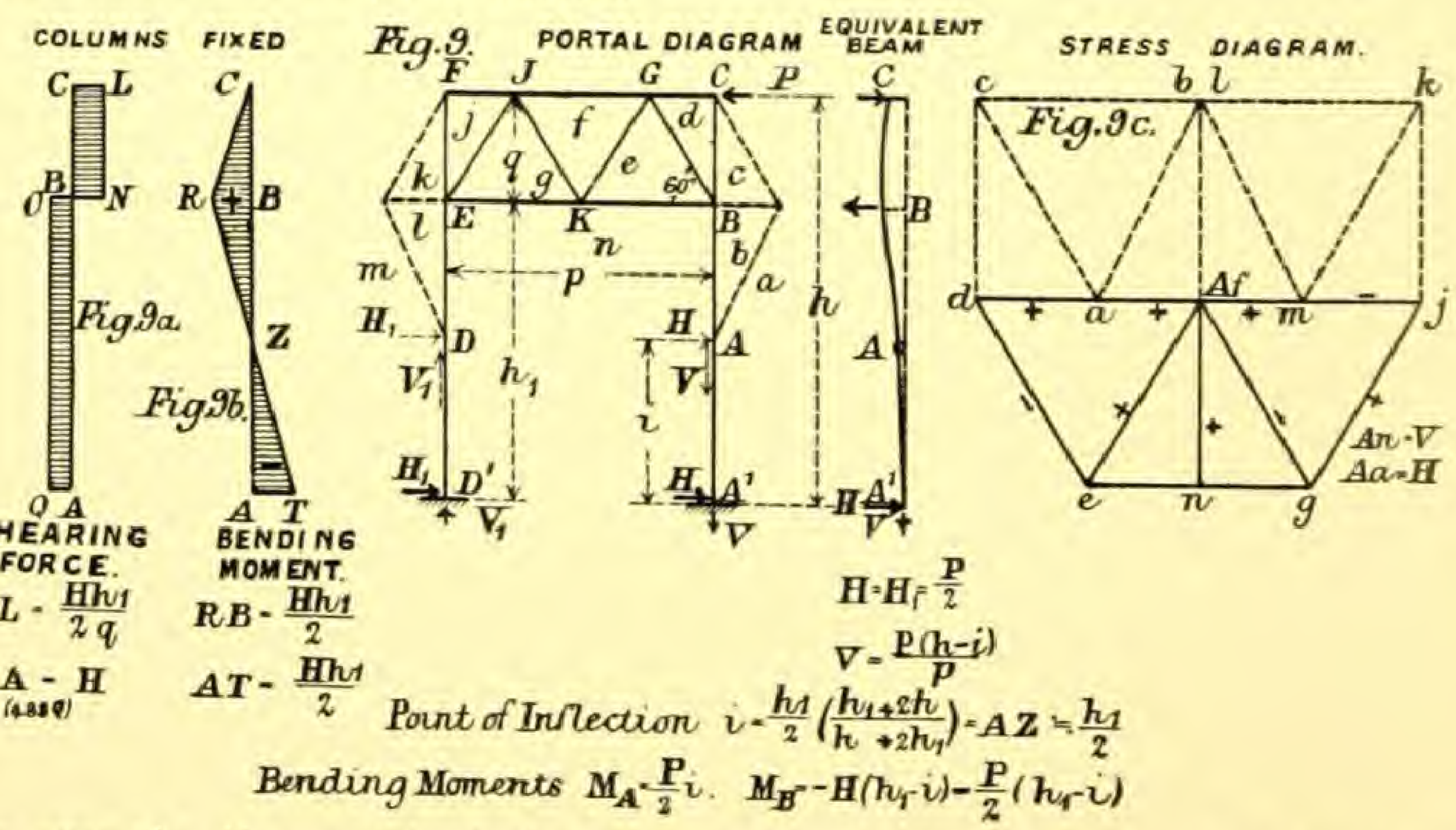
[1] "The Design of Steel Mill Buildings" (page 99), The Engineering News Publishing Company, New York, 1904.

II.—FIXED COLUMNS.

Any of the previous portals might be fixed at the base. They may be considered as fixed by virtue of the direct stress due to vertical loads, when $V \times \frac{1}{2} p \geqq M_{max.}$ at A or D with fixed ends ; $M_{max.} = \frac{Ph_1}{4}$ where V = the total stress in the column. Columns with ends fixed may be treated by the preceding formulæ for hinged columns by applying the reactions H and V at the point of inflection and proceeding as before. The correct distance of the point of contra-flexure from the fixed end is given by $i = \frac{h_1}{2}\left(\frac{h_1 + 2h}{h + 2h_1}\right)$. This is slightly greater than $\frac{h_1}{2}$, the value which is usually taken, and which is quite accurate enough for ordinary purposes. h then becomes $h - \frac{1}{2} h_1$, and consequently the portal stresses are reduced.

The moments at B and E will be one-half their previous value.

Also $M_A = M_D = -\frac{Ph_1}{4} = -M_B = -M_E$. If the resisting moment at A or D < $P\frac{h_1}{4}$, then M_A = resisting moment and point of inflection $i = \frac{2 M_A}{P}$. Then use $(h - i)$ instead of h in formulæ for portal stresses.



For example, if a portal of the type shown in Fig. 9 be fixed at the base, we shall have for stresses

$$S_{CG} = P\left\{1 + \frac{h_1 - i}{2q}\right\} \qquad S_{BE} = P\frac{h - i}{2q} \qquad S_{FB} = V \sec a = \frac{P(h - i)}{p} \frac{\sqrt{p^2 + q^2}}{q}$$

The anchorage can be obtained from the bending moment at the base of the column. It is a maximum on the windward side. Let W = direct stress from vertical loading, b = length of base plate of column, A = area of base in square inches, I = moment of inertia about axis at right angles to direction of wind, M = bending moment = $P\frac{h_1}{4}$, d = distance between the bolts, T = anchorage force.

Then

$$Td - (W - V)\frac{d}{2} + \frac{Ph_1}{4} = O \text{ or } T = \frac{1}{4d}\left[2(W - V)d - Ph_1\right],$$

and the greatest pressure on the masonry under the leeward edge of the plate is $C = \frac{W}{A} + \frac{Ph_1b}{8I}$. Fig. 9 is an example with fixed ends. The graphical stress diagram is obtained similarly to Fig. 8c.

III.—COLUMN BASES ON DIFFERENT LEVELS.

Fig. 10.—In this case H will not be equal to H_1, but will be calculated from the formulæ given under Fig. 10. Each column should be treated separately by first calculating its H, then the point of application of H or point of inflection from formula (Fig. 9), when the shearing force and bending moment diagrams can be got out for each. They will be similar to Figs. 9A and 9B. If there be a plate girder from B to C equivalent to fixing the columns at B, then

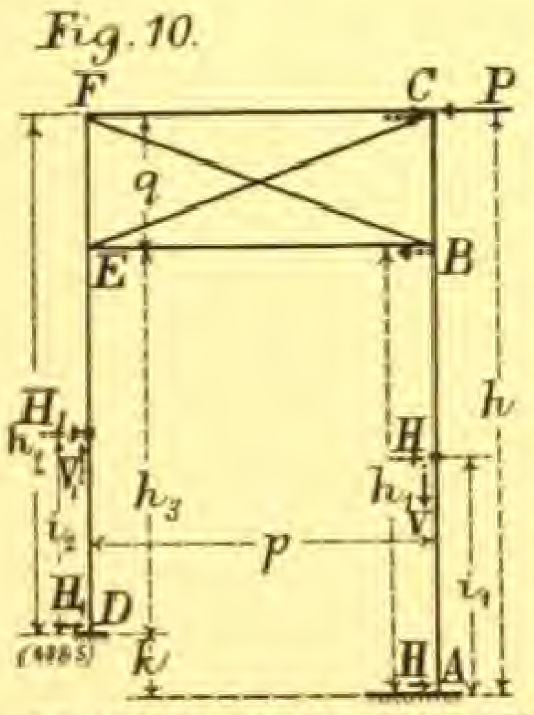
$$q = 0 \qquad H_1 = \frac{PI_2h_1^3}{I_2h_1^3 + I_1h_3^3} \text{ and } i_2 \doteqdot \frac{h^3}{2}.$$

This case can be applied to an elevated railway.

Note.—The formulæ given for Fig. 10 are based on the assumption that $\frac{H}{H_1} = \frac{\delta_B}{\delta_E}$ where δ is the deflection at the point indicated by the suffix attached to it.

$$V_1 = -V = \frac{H_1(h_2 - i_2) + H(h - i_1)}{p}.$$

Fig. 10.



I_1 – Moment of Inertia of Column A C.

I_2 – Moment of Inertia of Column D F.

$$i_1 = \frac{h_1}{2}\left(\frac{h_1 + 2h}{h + 2h_1}\right)$$

$$i_2 = \frac{h_3}{2}\left(\frac{h_3 + 2h_2}{h_2 + 2h_3}\right)$$

$$H_1 = \frac{P}{1 + \frac{I_1h_3^3}{I_2h_1^3}\left\{\frac{(3h_3+4q)(3h_1+q)}{(3h_1+4q)(3h_3+q)}\right\}}$$

$$H = P - H_1$$

COLUMNS ON DIFFERENT LEVELS.

IV.—CONTINUOUS PORTALS.

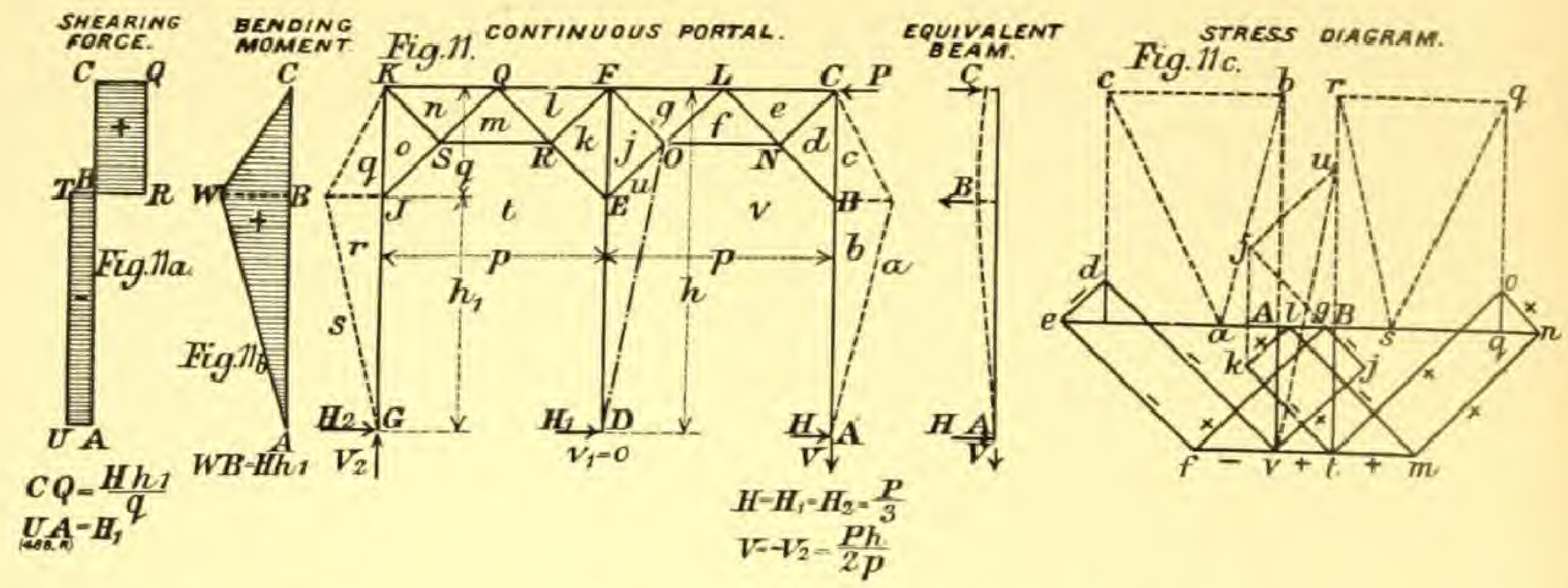
If N be the number of bays, then $H = H_1 = H_2 = H_n = \frac{P}{N+1}$. Consider any column. Let g be its distance from G the centre of gravity of the columns. Then if V and H be the reactions, and M the bending moment, $V \propto g$ and $H \propto g$. But $M \propto g^2$. Again, let u be the reaction at unit distance from G. Then

$$M = u\,(g_1^2 + g_2^2 + g_3^2 + \cdots\cdots g_n^2) = Ph$$

$$u\,\Sigma\,g^2 = Ph \text{ or } u = \frac{Ph}{2\,g^2}.$$

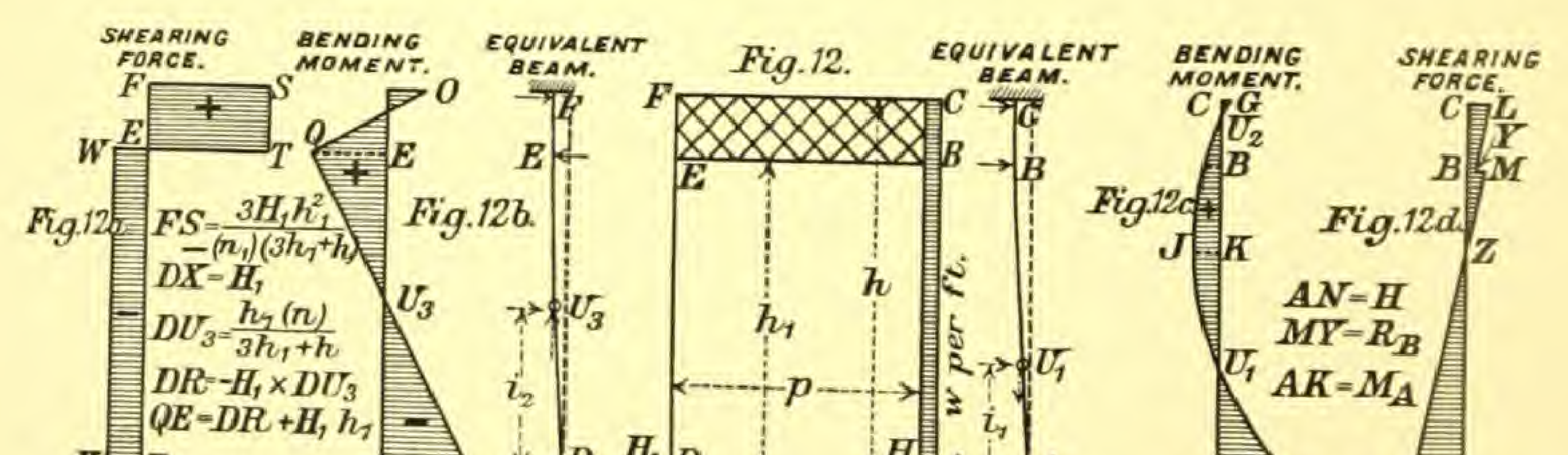
First find u, then the reactions and stresses.

Note that the columns on the windward side of G will be in tension, and those on the leeward side in compression. In Fig. 11 two bays have been taken, and the diagrams are given as usual. The bending moment and shearing force diagrams are the same for all three columns. The method of Fig. 11c is also due to Ketchum.



V.—UNIFORMLY DISTRIBUTED LOAD ON PORTAL.

It is more correct in the case of mill buildings, etc., to take the load on the windward portal as uniformly distributed. This has been done in Fig. 12.



Leeward Diagrams. *Ends assumed fixed at A, C, D & F.* *Windward Diagrams.*

Let $(m) = h^5 + 4h^4 h_1 + 4h^3 h_1^2 + 6h^2 h_1^3 + 3h h_1^4$ Let $(n) = h_1 + h$

$(m_1) = -3h^3 - 3h^2 h_1 + h h_1^2 - h_1^3$ $(n_1) = h_1 - h$

$$R_B = \frac{wh\,(m)}{36 h_1^2 (m_1)} \qquad H = \frac{6 R_B (n_1^2) + h (n^2)}{2h (2h_1 + h)} \qquad H_1 = wh - H \qquad V_1 = -V = \frac{H_1(h - i_2) + H(h - i_1)}{p} - \frac{w(h-i_1)^2}{2p}$$

$$M_A - AK = -H\frac{h}{2} + \frac{R_B}{2h}(n_1^2) + \frac{wh^2}{6} \qquad AU_1 = H - \frac{1}{w}\sqrt{H^2 + 2M_A w}$$

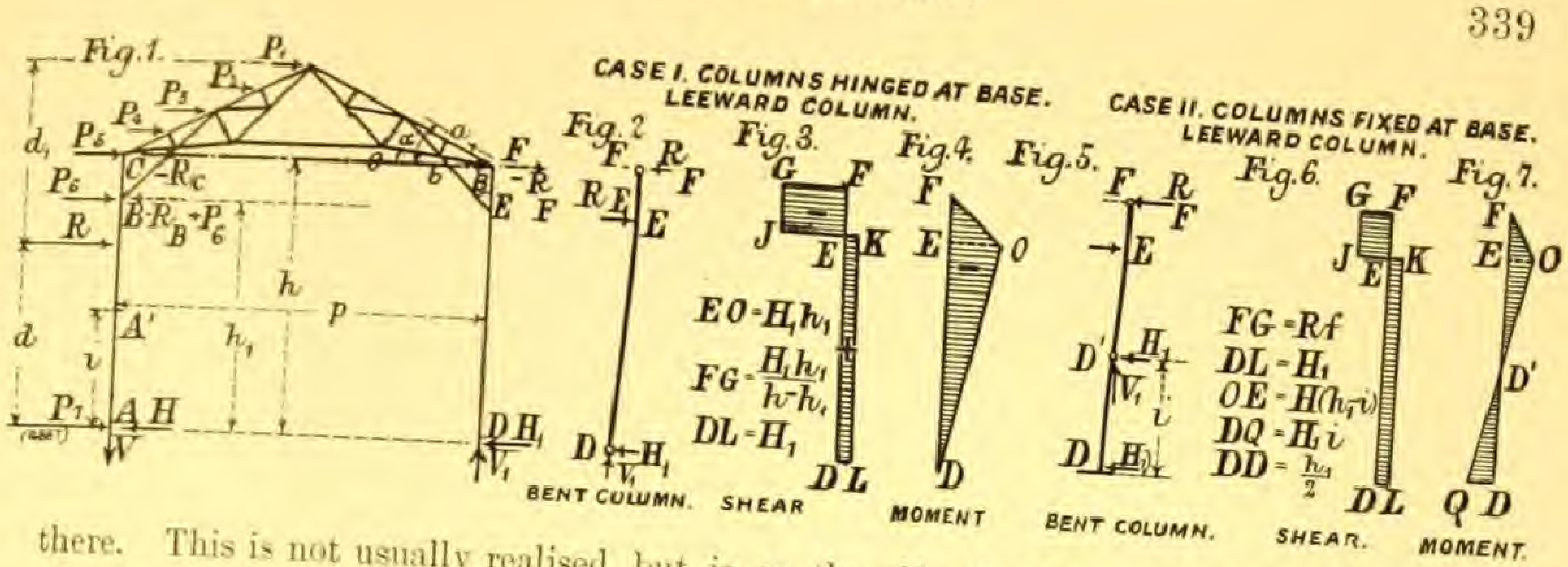
Approximate Solution $H = \frac{w}{16}(6h + 5h_1)$ $H_1 = \frac{w}{16}(10h - 5h_1)$

Then substitute this value of H_1 in all expressions given for leeward diagrams since leeward column being worst need only be considered. Values will be correct to 3 per cent for ordinary types.

COLUMNS SUPPORTING ROOFS.

Bending and Deflection due to wind loads, shafts, hoists, cranes or any loads which can be resolved into horizontal and vertical components.

A French truss roof such as is commonly used for 50 ft. or 60 ft. spans is shown, supported by columns, in Fig. 1. The connection between the truss and column can usually be reduced to a line like b E. The following treatment applies to any loads as mentioned above, but it is wind pressure which is kept particularly in view. When the upper ends of the columns are assumed hinged to the truss there is no bending moment



there. This is not usually realised, but is on the side of safety. The lower ends may be taken as hinged or fixed. In practice the condition is perhaps usually between the two, but it is generally better to take them as hinged. In the example given the wind pressure acts uniformly on one side, the left side. The resultant pressures are taken acting at the panel points. $P_2 = P_3 = P_4 = 2\,P_1 =$ wind load per panel. $P_5 = P_1 + \frac{1}{2}$ (wind on BC). The results of Example V. opposite may be applied to this case.

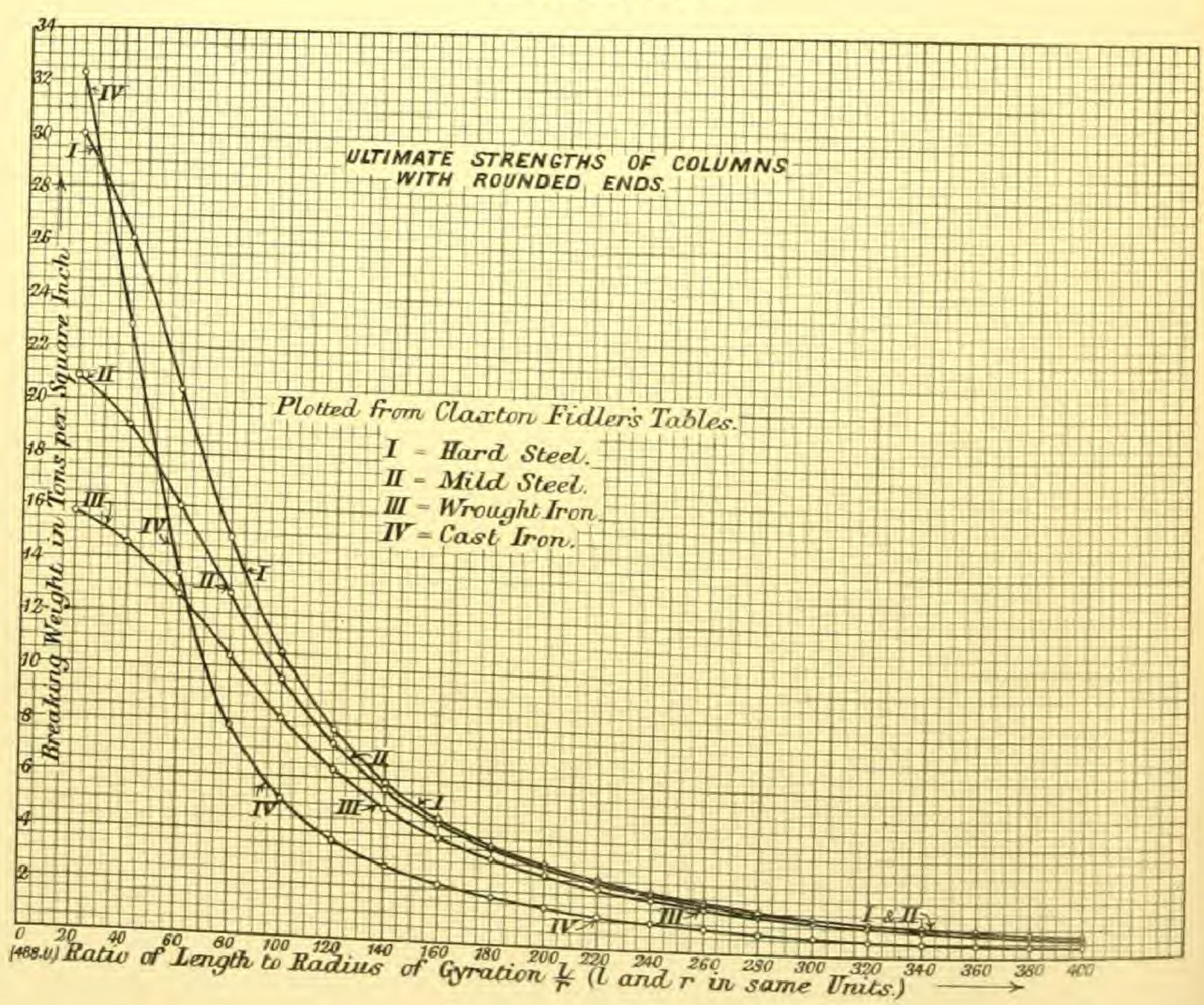
Case I.—Columns hinged at base and top. $P_6 = \frac{1}{2}$ (wind on BC + wind on AB). $P_7 = \frac{1}{2}$ (wind on AB). $H = H_1 = \dfrac{\Sigma P}{2} = \dfrac{R}{2}$, where R is the resultant of all the wind loads. $V_1 = -V = \dfrac{Rd}{p}$ by moments of external forces about F. These are the direct vertical stresses on the columns due to wind. For the windward side we have to subtract the wind pressure at A, B and C from H, R_B (the force at B) and R_C (the reaction at C) respectively. The bending moment at B, $M_B = (H - P_7)h_1$. The maximum bending will take place at E, the foot of the knee bracket, on the leeward side $M_E = H_1 h_1$. The maximum fibre stress $f_{max.}$ due to wind and direct loading will also occur there, compression inside and tension outside. Due to direct loading, W, $f_2 = \dfrac{W}{A}$. Due to flexure, $f_1 = \dfrac{My}{I \pm \dfrac{Wh^2}{10\,E}}$, the units being pounds and inches. For explanation of this formula see page 371, Combined Stresses. $f_{max.} = f_2 \pm f_1$. The diagrams shown are for the leeward side only, since the bending is greater. Again by moments about E, $R_F = \dfrac{H_1 h_1}{h - h_1}$ (Fig. 2) and $R_E = H_1 + R_F$. The following are the stresses in bE, aF and bF (Fig. 1) by resolution. $S_{bE} = R_E \operatorname{cosec} \beta$. $S_{aF} = (V - R_E \operatorname{cotan} \beta) \operatorname{cosec} \alpha$. $S_{bF} = (R_F - S_{aF} \cos \beta) \cos \theta$. Knowing these stresses, the stress diagram for the truss can at once be drawn in the usual way.

Case II.—Columns fixed at base, hinged at top. This case is best treated similarly to Case I. by applying H and V at the point of inflection, instead of at the base. Note that P_6 is now $= \frac{1}{2}$ (wind on BC + wind on A′B) where A′ is the point of contra-flexure. $AA' = i = \dfrac{h_1}{2}\left(\dfrac{h_1 + 2\,h}{h + 2\,h_1}\right) \doteq \dfrac{h_1}{2}$. The assumption involved in the integration for i, and the results which follow, is that the deflections at E and F are equal, the leeward column again being treated. $H = H_1 = \dfrac{R}{2}$ $V = -V_1 = \dfrac{1}{2\,p}(2\,H - wi)(2\,d - i)$.

For the windward column, the shear at B, $R_B = H\dfrac{(h - i)}{h - h_1}$ and $M_A = Hi - \dfrac{wi^2}{2}$ where w is the wind pressure per foot of height of column. For the leeward column, the shear at F, $R_F = \dfrac{H_1}{2}\dfrac{3h_1^2}{(h - h_1)(h + 2h_1)}$, and $R_E = H_1 + R_F$. $M_D = H_1 i$, and is the maximum negative moment. The maximum positive occurs at E, and is $M_E = H(h_1 - i)$. The maximum fibre stress also occurs at E, and is $f_2 \pm f_1 = \dfrac{W}{A} \pm \dfrac{My}{I \pm \dfrac{W(h - i)^2}{10E}}$. The formulæ of Case I. will give the stresses in members meeting at E and F.

Case III.—Columns fixed at both base and top. The point of inflection will obviously be at $i = \dfrac{h_1}{2}$. Other values will be as follows for leeward column:— $M_E = \dfrac{H_1}{2}h_1$. $R_F = \dfrac{H_1 h_1}{2(h - h_1)}$. $R_E = H_1 + R_F$. The formulæ in Case I. will apply for stresses.

Columns.



ULTIMATE STRENGTHS OF COLUMNS.

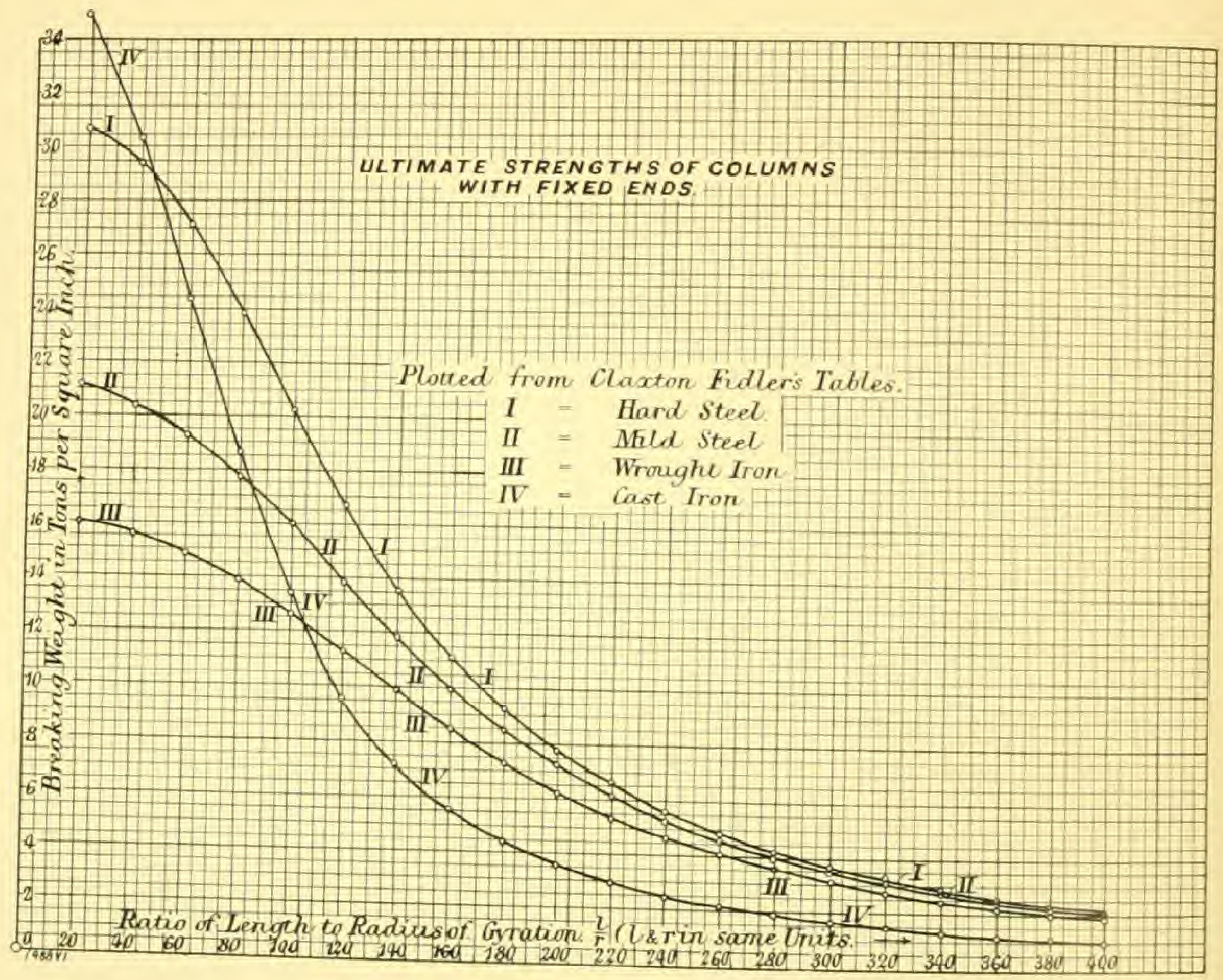
THE following tables are taken from "A Practical Treatise on Bridge Construction," by T. C. Fidler, M.I.C.E. (Chapter X.), and the values have been reduced to tons per square inch.

l = Length of column.

r = Radius of gyration in same units as l.

I.—*Rounded Ends.* Breaking weight in tons per square inch of sectional area.

$\frac{l}{r}$.	Cast Iron.	Wrought Iron.	Mild Steel.	Hard Steel.
20	32.3	15.7	20.8	30.0
40	22.7	14.5	19.1	26.1
60	13.4	12.7	16.1	20.3
80	7.9	10.3	12.6	14.7
100	5.2	8.1	9.6	10.6
120	3.7	6.3	7.3	7.8
140	2.8	5.0	5.7	5.9
160	2.2	3.9	4.5	4.6
180	1.74	3.2	3.6	3.7
200	1.43	2.63	3.0	3.0
220	1.20	2.22	2.5	2.55
240	1.01	1.88	2.1	2.15
260	0.87	1.62	1.82	1.85
280	0.75	1.40	1.58	1.59
300	0.66	1.23	1.38	1.40
320	0.58	1.08	1.22	1.22
340	0.52	1.04	1.08	1.09
360	0.46	0.87	0.98	0.98
380	0.42	0.77	0.87	0.88
400	0.38	0.70	0.78	0.79



The formula from which the breaking weights have been calculated is

$$p = \frac{\rho + f - \sqrt{(\rho + f)^2 - 4 f \rho (1 - \phi)}}{2 (1 - \phi)}$$

where p is the load that would produce any given stress f. f = ultimate compressive strength of the material. See further "A Practical Treatise on Bridge Construction," Chapter X. For columns with fixed ends l is taken as equal to $\frac{6}{10}$ L, where L is the total length of the strut, and l the length of the equivalent round-ended strut.

II.—*Fixed Ends.* Breaking weight in tons per square inch of sectional area.

$\frac{l}{r}$.	Cast Iron.	Wrought Iron.	Mild Steel.	Hard Steel.
20	34.7	16.0	21.1	30.7
40	30.3	15.6	20.4	29.4
60	24.4	14.9	19.3	27.0
80	18.7	13.9	17.8	23.9
100	13.4	12.7	16.1	20.3
120	9.5	11.3	13.8	16.7
140	7.1	9.9	11.8	13.6
160	5.6	8.6	10.0	11.2
180	4.6	7.4	8.5	9.3
200	3.7	6.3	7.3	7.8
220	3.1	5.4	6.2	6.7
240	2.54	4.7	5.4	5.6
260	2.23	4.15	4.6	4.9
280	1.96	3.66	4.1	4.2
300	1.74	3.21	3.66	3.75
320	1.52	2.81	3.21	3.26
340	1.34	2.50	2.81	2.90
360	1.20	2.27	2.45	2.54
380	1.10	2.05	2.27	2.32
400	1.01	1.88	2.12	2.14

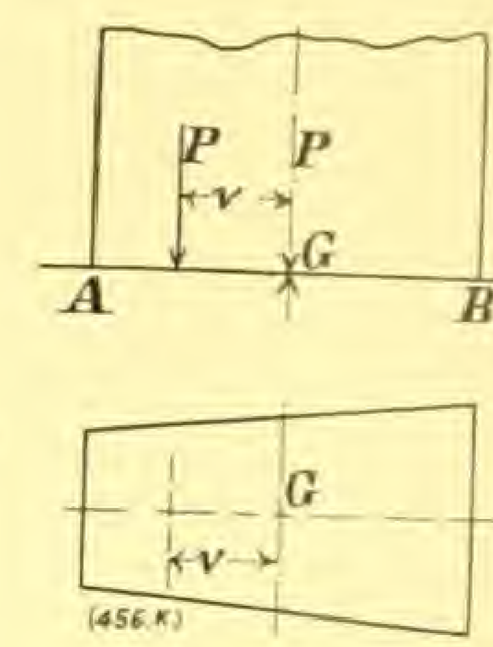
The following are the values of f, the ultimate compressive stress in pounds per square inch, assumed in calculating the foregoing.

Hard Steel.	Mild Steel.	Wrought Iron.	Cast Iron.
70,000	48,000	36,000	80,000

Eccentric Loads.

THE distribution of stress on a section when acted upon by a normal force P, the line of action of which does not pass through the centre of gravity of the section.

Let P = The normal force
G = Centre of gravity of the section.
v = Distance of the force P from G.
A = Area of section.
I = Moment of inertia of section.
r = Radius of gyration of section.
f = Intensity of pressure at distance y from G.
y = Distance from G of the pressure f.



Assume two equal and opposite forces to be applied to the section at its centre of gravity, both equal to P. The condition of equilibrium is not thus affected. The section is now acted upon by a force P at its centre of gravity producing uniform stress $\frac{P}{A}$, and a couple, the moment of which is Pv.

As $M = \frac{fI}{y}$, the stress produced by the moment Pv at a point distant y from G is $\frac{Pv}{I}y$. Therefore the total stress at this point is

$$f = \frac{P}{A} \pm \frac{Pv}{I}y$$

and as $I = Ar^2$, therefore

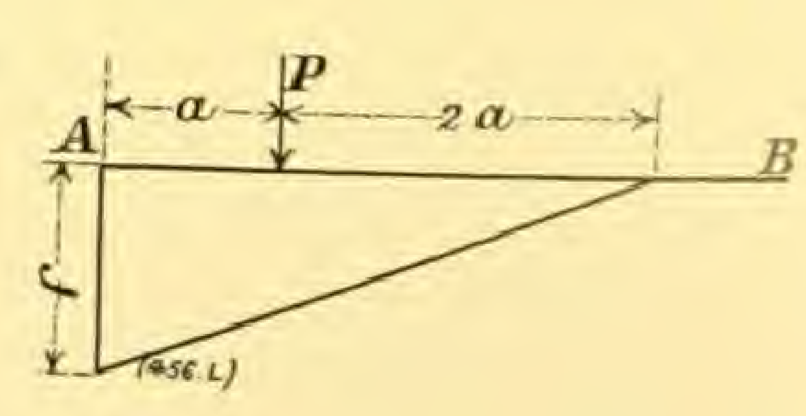
$$f = \frac{P}{A}\left\{1 \pm \frac{vy}{r^2}\right\}$$

the plus or minus sign being taken according to whether the stress due to the moment is of the same kind or of opposite kind to $\frac{P}{A}$

For calculating the stresses on masonry, concrete, etc., materials incapable (except to a very limited extent) of resisting tension, the above formula can only be applied as long as v does not exceed $\frac{r^2}{y}$. For rectangular sections, therefore, v must not exceed one-sixth of the base AB. Where v exceeds $\frac{r^2}{y}$, let a = the distance of P from A or B, then the length of the section under pressure is $3a$, since P must pass through the centre of pressure. The maximum compression at A will therefore be

$$f = \frac{2P}{A'},$$

where A' is the area of the section whose width is $3a$.



General Formulæ

for Moment of Inertia, Radius of Gyration, etc., of Beams, Shafts, and Various Sections.

Let A = Area of the section

y = Distance of extreme fibre from an axis through the centre of gravity (neutral axis)

I_{xx} = Moment of inertia of section about its neutral axis

I_{aa} = Moment of inertia of section about another line distant "d" from the neutral axis, and parallel to it

Z = Modulus of section

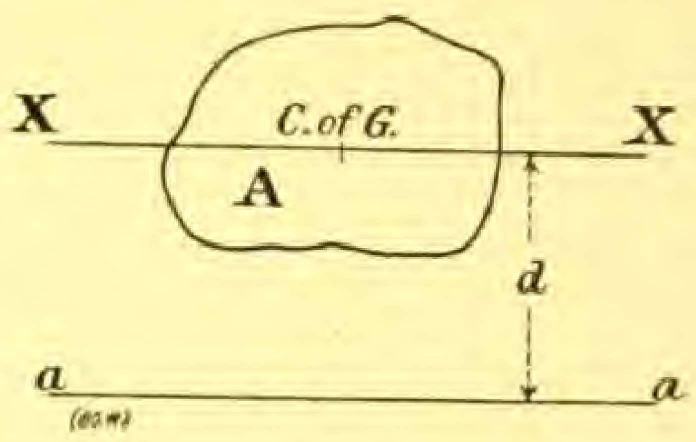
r = Radius of gyration of section

M = Bending moment at the section (in beams, etc.)

T_m = Twisting moment at the section (in shafts, etc.)

R = Moment of resistance of section

f = Stress on extreme fibre at distance "y" from neutral axis



Then $I_{aa} = I_{xx} + A\,d^2$. $Z = \frac{I_{xx}}{y} = \frac{M}{f}$. $r = \sqrt{\frac{I_{xx}}{A}}$ $M = R = f\frac{I}{y} = fZ$

$$T_m = R = f\frac{I}{y} = fZ*$$

ROUTH'S RULE.—It applies only to bodies having three perpendicular semi-axes of symmetry.

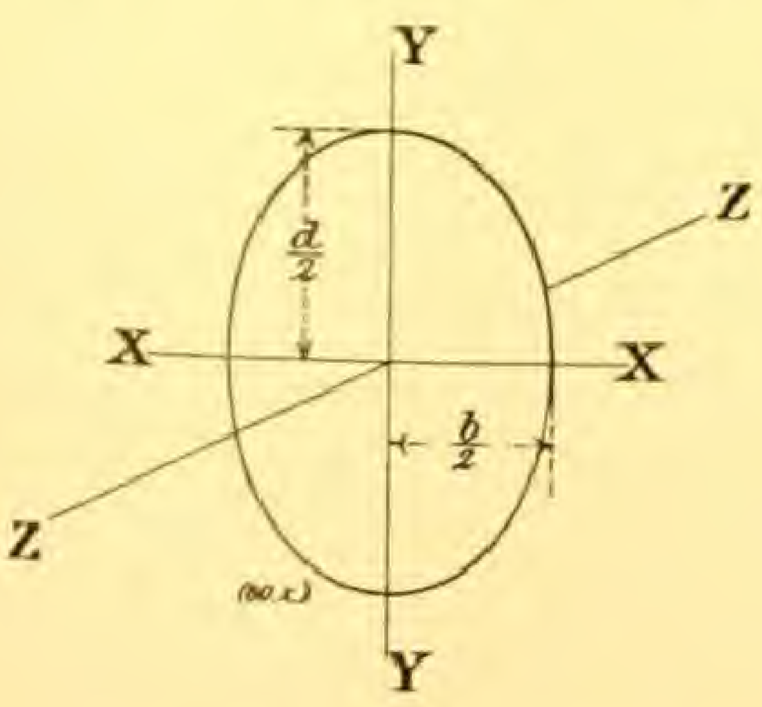
$$\text{Moment of Inertia} = \frac{\text{Mass Volume or Area} \times \text{sum of squares of perpendicular semi-axes of symmetry}}{3,\ 4 \text{ or } 5}$$

Denominator = 3 for rectangular bodies

Denominator = 4 for elliptical bodies

Denominator = 5 for ellipsoidal bodies.

For example, take an elliptical area:

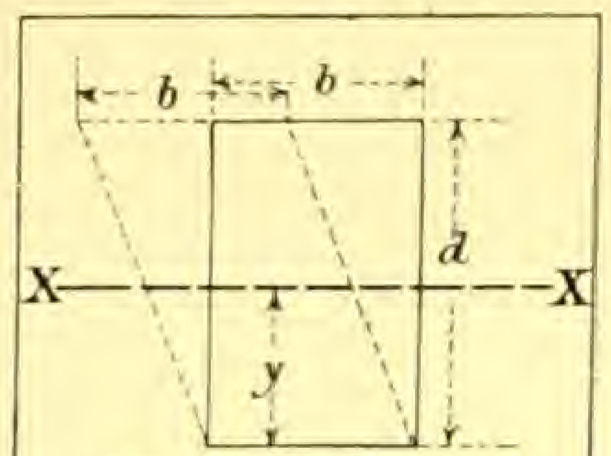
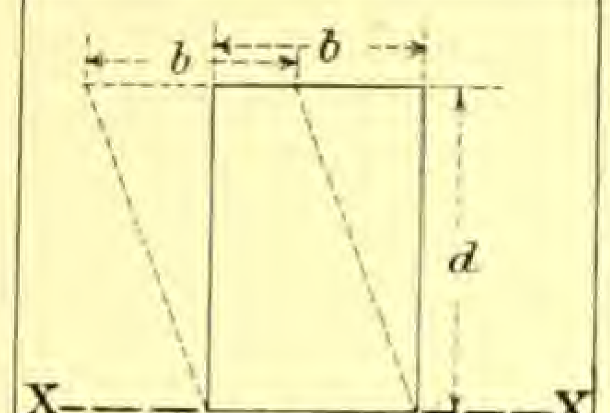
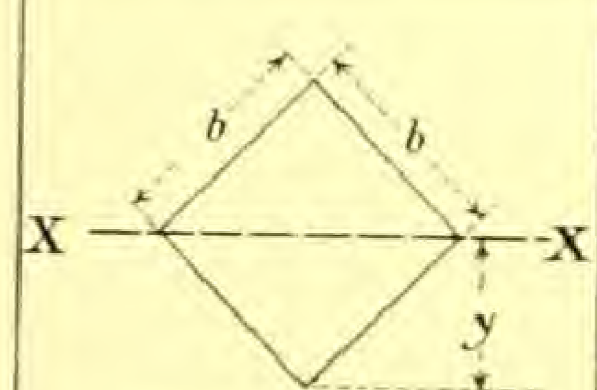
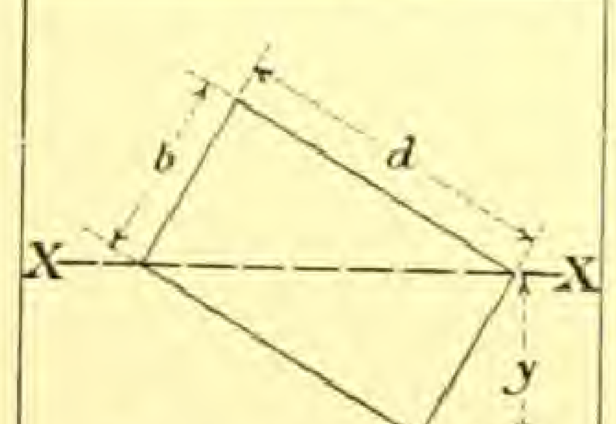
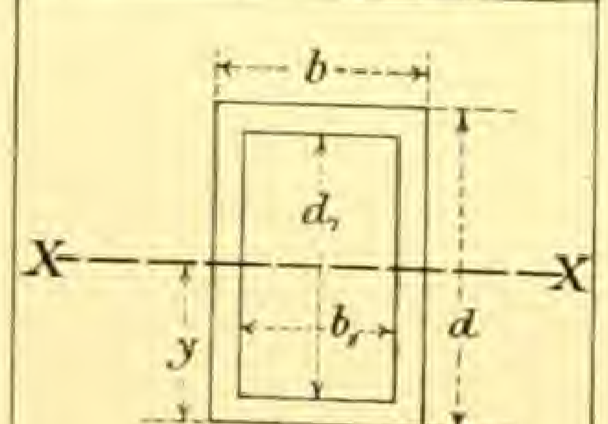
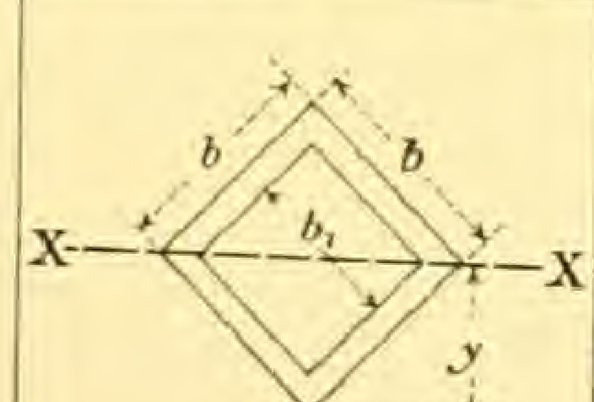


$$I_{xx} = \frac{\pi b d}{4} \times \frac{\left(\frac{d}{2}\right)^2 + 0}{4} = \frac{\pi b d^3}{64}$$

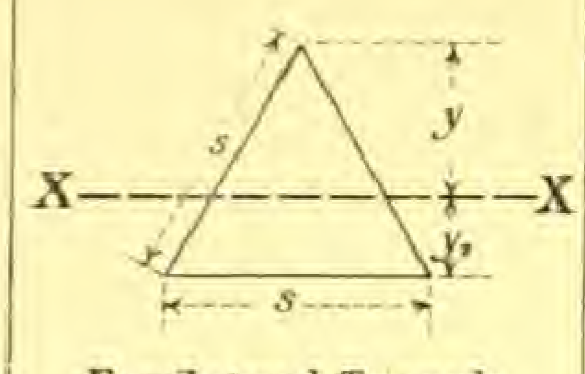
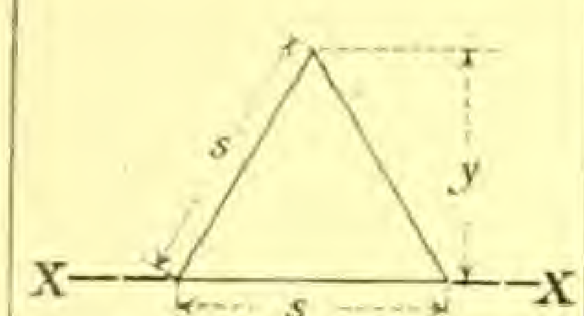
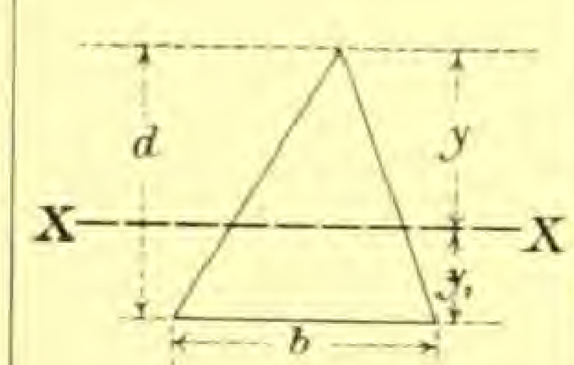
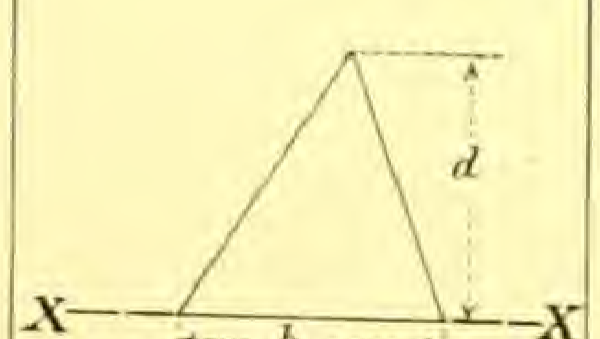
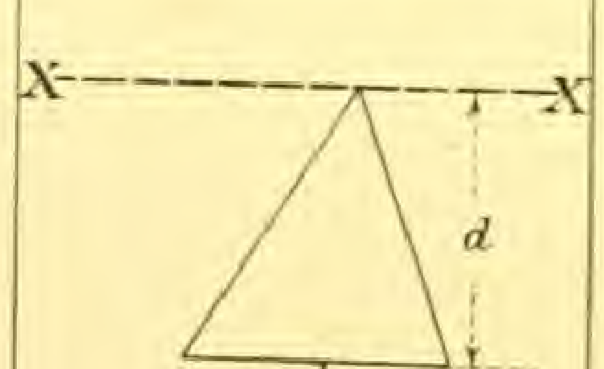
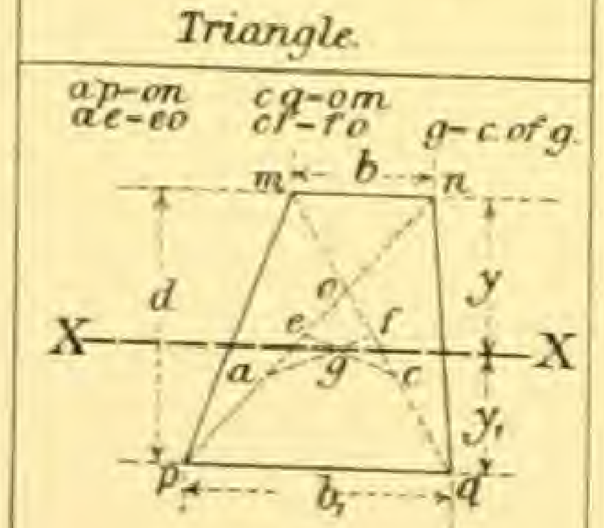
$$I_{yy} = \frac{\pi b d}{4} \times \frac{\left(\frac{b}{2}\right)^2 + 0}{4} = \frac{\pi d b^3}{64}$$

$$I_{zz} = \frac{\pi b d}{4} \times \frac{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}{4} = \frac{\pi b d (b^2 + d^2)}{64}$$

* There are restrictions to the use of this formula. *Vide*, for example, Lineham's "Mechanical Engineering," page 421, Fifth Edition.

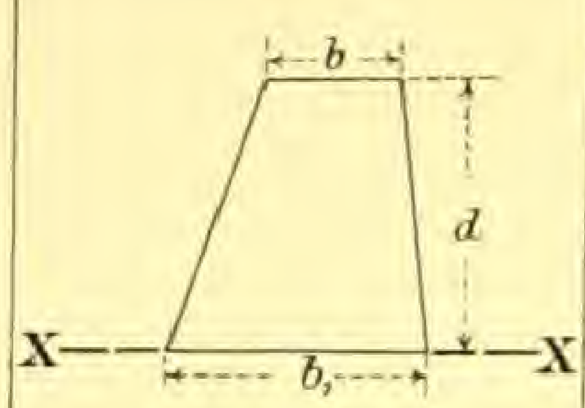
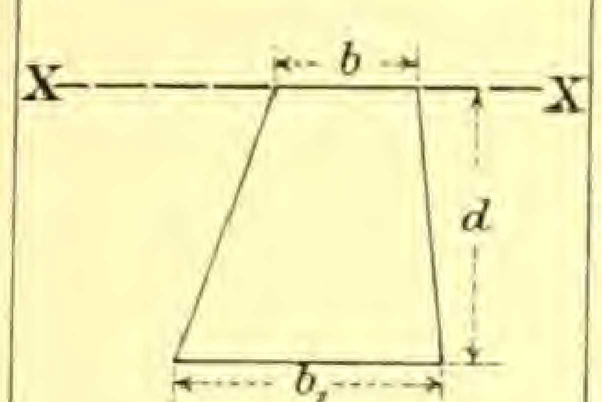
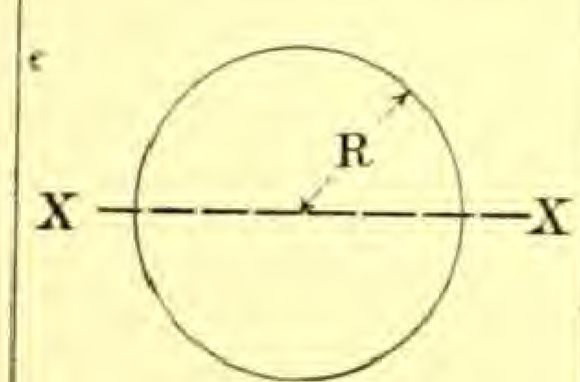
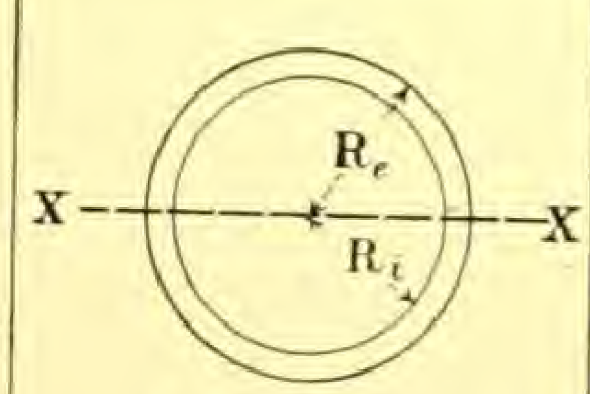
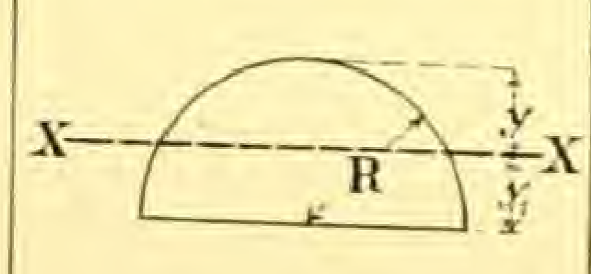
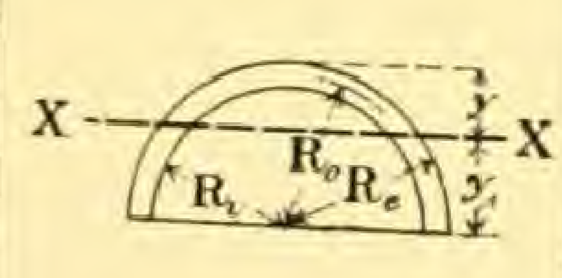
Section.	Area, A	Distance of Neutral Axis from Extreme Fibres. y and y_1.
 *Square, Rectangle or Rhombus.*	$b\,d$	$\frac{d}{2}$
 *Square, Rectangle or Rhombus.*	$b\,d$	d
 *Square on edge*	b^2	$\frac{b}{\sqrt{2}} = 0.707\,b$
 *Rectangle on edge.*	$b\,d$	$\frac{b\,d}{\sqrt{b^2 + d^2}}$
 *Hollow Square or Rectangle.*	$b\,d - b_1\,d_1$	$\frac{d}{2}$
 *Hollow Square on edge.*	$b^2 - b_1^2$	$\frac{b}{\sqrt{2}} = 0.707\,b$

Moment of Inertia about xx. I_{xx}.	Section Modulus. $z = \frac{I}{y}$	Radius of Gyration. $r = \sqrt{\frac{I}{A}}$.
$\frac{b\,d^3}{12}$	$\frac{b\,d^2}{6}$	$\frac{d}{\sqrt{12}} = 0.2887\,d$
$\frac{b\,d^3}{3}$	$\frac{b\,d^2}{3}$	$\frac{d}{\sqrt{3}} = 0.5774\,d$
$\frac{b^4}{12}$	$0.118\,b^3$	$\frac{b}{\sqrt{12}} = 0.2887\,b$
$\frac{b^3\,d^3}{6\,(b^2 + d^2)}$	$\frac{b^2\,d^2}{6\,\sqrt{b^2 + d^2}}$	$0.4083\,\frac{b\,d}{\sqrt{b^2 + d^2}}$
$\frac{b\,d^3 - b_1\,d_1^3}{12}$	$\frac{b\,d^3 - b_1\,d_1^3}{6\,d}$	$\sqrt{\frac{b\,d^3 - b_1\,d_1^3}{12\,(b\,d - b_1\,d_1)}}$
$\frac{b^4 - b_1^4}{12}$	$\frac{\sqrt{2}}{12}\,\frac{b^4 - b_1^4}{b} \neq 0.1178\,\frac{b^4 - b_1^4}{b}$	$\sqrt{\frac{b^4 - b_1^4}{12\,(b^2 - b_1^2)}}$

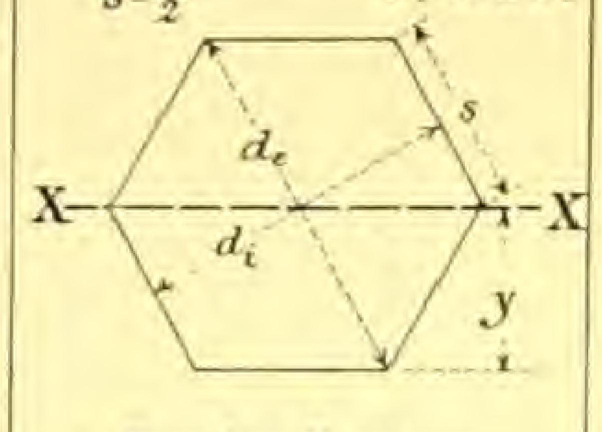
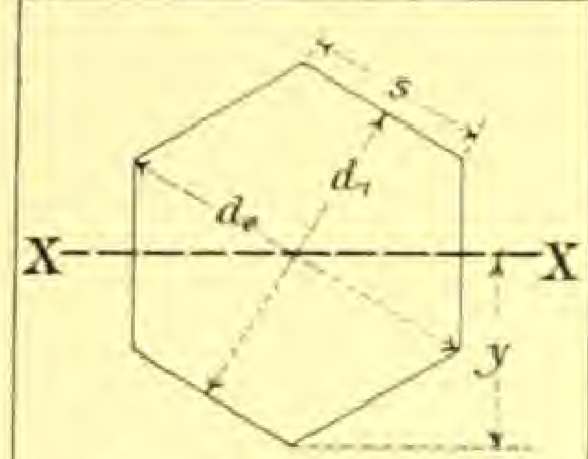
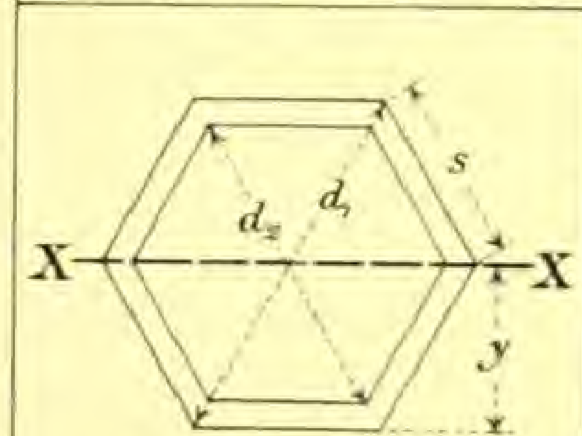
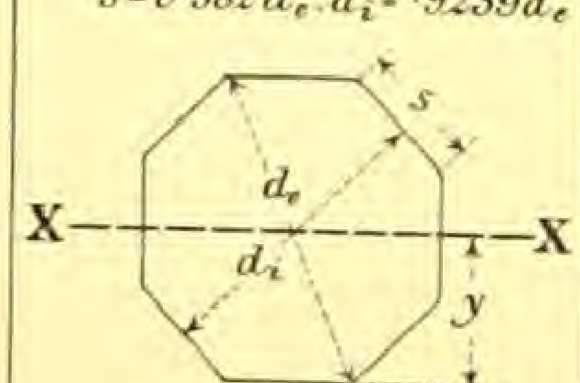
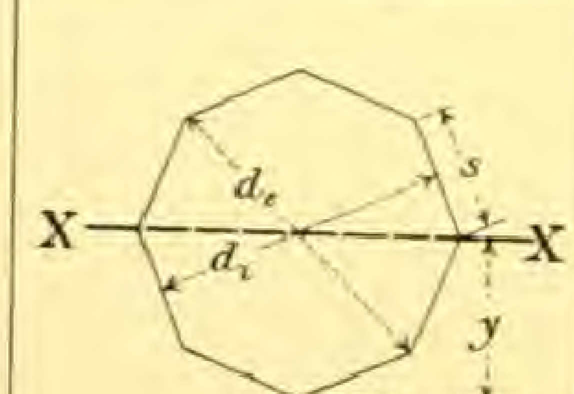
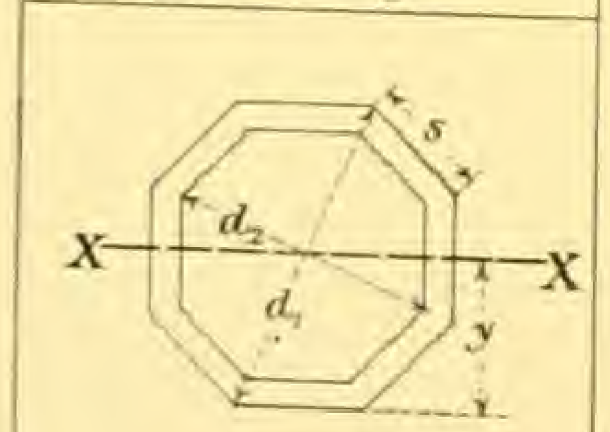
Section.	Area, A.	Distance of Neutral Axis from Extreme Fibres. y and y_1.
 Equilateral Triangle.	$\frac{\sqrt{3}}{4} S^2 = 0.433\ S^2$	$y = 0.5774\ S$ $y_1 = 0.2887\ S$
 Equilateral Triangle.	$\frac{\sqrt{3}}{4} S^2 = 0.433\ S^2$	$0.866\ S$
 Triangle.	$\frac{b\ d}{2}$	$y = \frac{2}{3} d$ $y_1 = \frac{1}{3} d$
 Triangle.	$\frac{b\ d}{2}$	d
 Triangle.	$\frac{b\ d}{2}$	d
 Trapezoid.	$\frac{b + b_1}{2} d$	$y = \frac{b + 2 b_1}{b + b_1} \frac{d}{3}$ $y_1 = \frac{b_1 + 2 b}{b + b_1} \frac{d}{3}$

(2438)

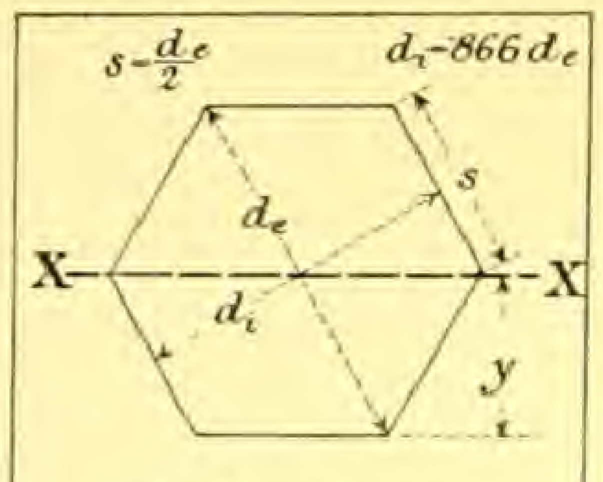
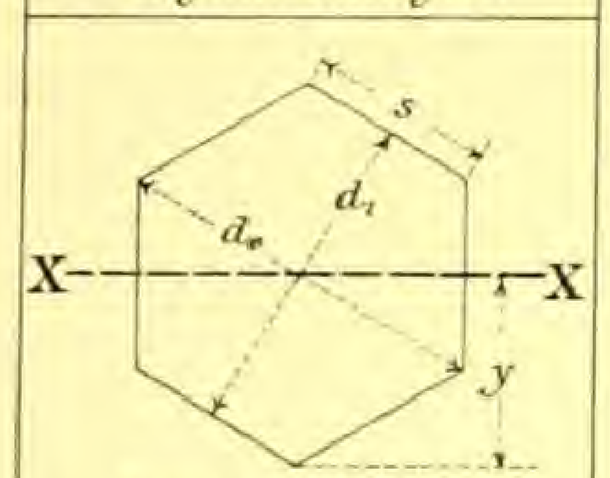
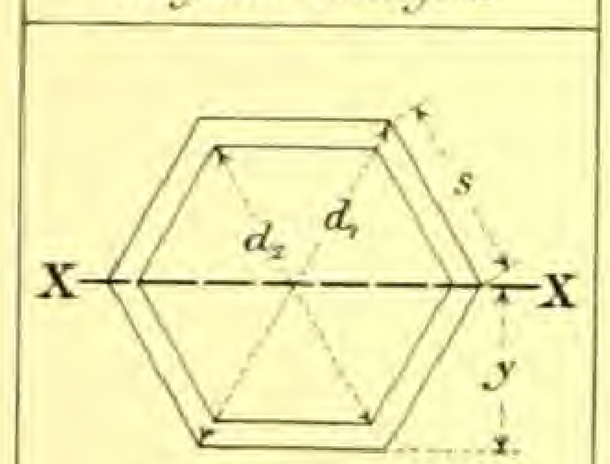
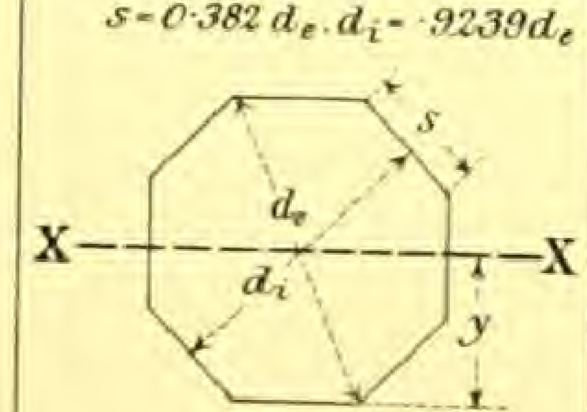
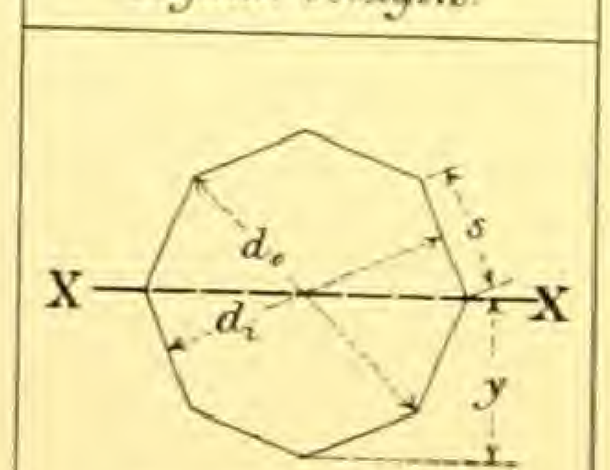
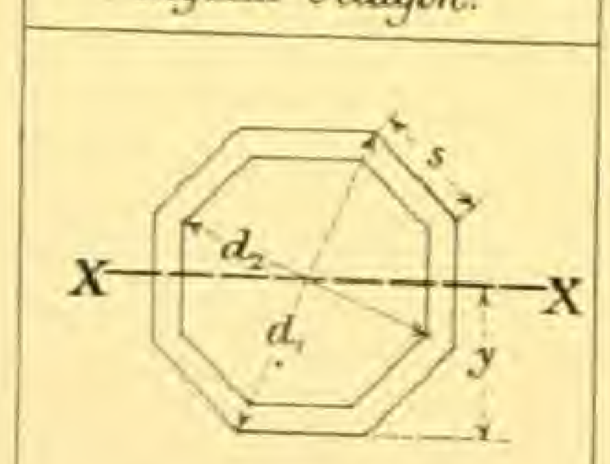
Moment of Inertia about $x x$. I_{xx}.	Section Modulus. $z = \frac{I}{y}$.	Radius of Gyration $r = \sqrt{\frac{I}{A}}$.
$0.018\,S^4$	$0.0311\,S^3$	$0.2039\,S$
$0.0541\,S^4$	$0.0625\,S^3$	$0.3536\,S$
$\frac{b\,d^3}{36}$	$\frac{b\,d^2}{24}$	$\frac{d}{\sqrt{18}} = 0.236\,d$
$\frac{b\,d^3}{12}$	$\frac{b\,d^2}{12}$	$\frac{d}{\sqrt{6}} = 0.4083\,d$
$\frac{b\,d^3}{4}$	$\frac{b\,d^2}{4}$	$\frac{d}{\sqrt{2}} = 0.707\,d$
$\frac{b^2 + 4\,b\,b_1 + b_1^2}{36\,(b + b_1)}\,d^3$	$\frac{b^2 + 4\,b\,b_1 + b_1^2}{12\,(b + 2\,b_1)}\,d^2$	$\frac{d}{6\,(b+b_1)}\sqrt{2\,(b^2 + 4\,b\,b_1 + b_1^2)}$

Section.	Area, A.	Distance of Neutral Axis from Extreme Fibres. y and y_1.
 *Trapezoid.*	$(b+b_1)\frac{d}{2}$	d
 *Trapezoid.*	$(b+b_1)\frac{d}{2}$	d
 *Circle.*	πR^2	R
 *Hollow Circle*	$\pi(R_e^2 - R_i^2)$	R_e
 *Semicircle.*	$\frac{\pi R}{2}$	$y_1 = 0.4244 R$
 *Hollow Semicircle.*	$\frac{\pi}{2}(R_e^2 - R_i^2)$	$y = R_e - y_1$ $y_1 = \frac{4}{3\pi}\frac{R_e^2 + R_e R_i + R_i^2}{R_e + R_i}$

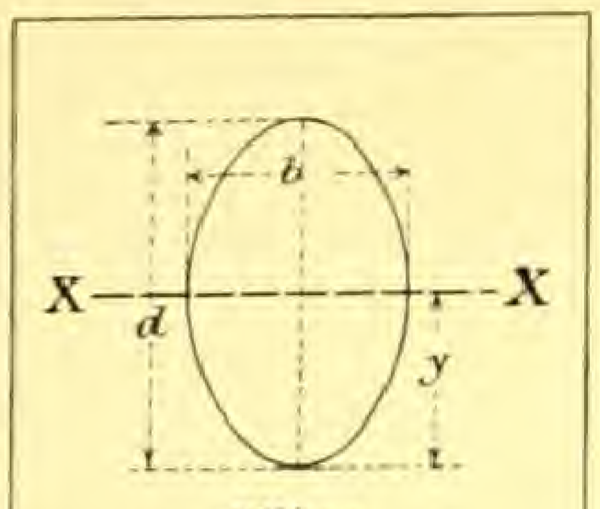
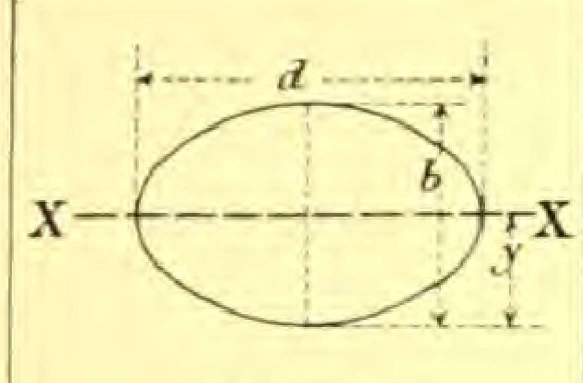
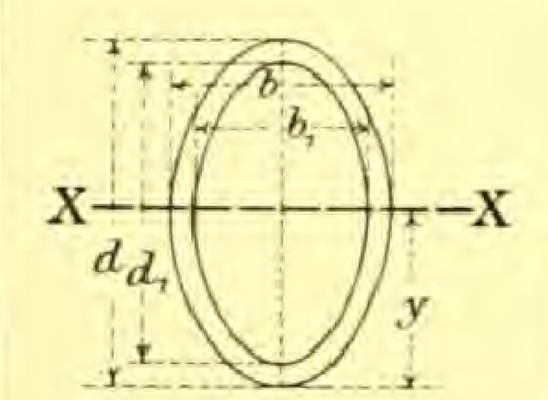
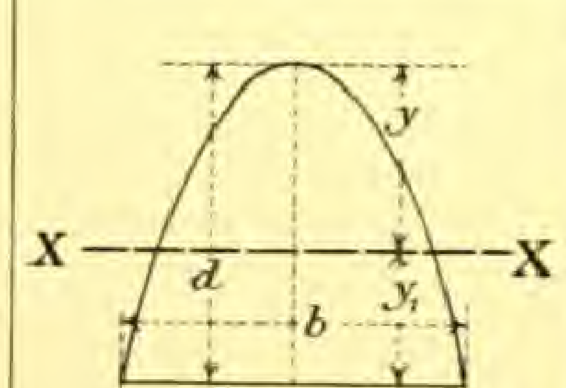
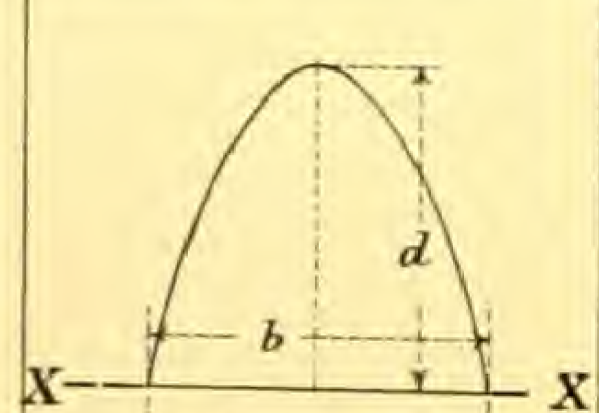
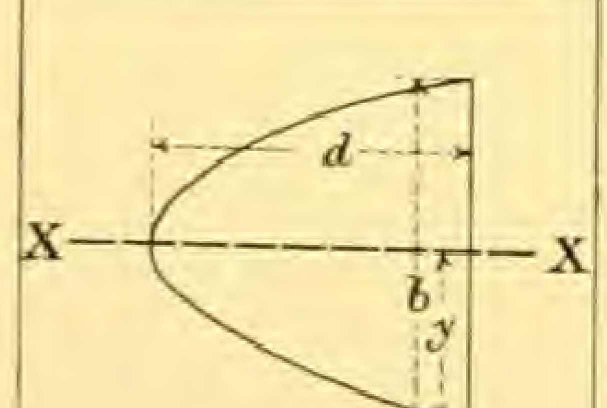
Moment of Inertia about xx. I_{xx}	Section Modulus. $z = \frac{I}{y}$.	Radius of Gyration. $r = \sqrt{\frac{I}{A}}$.
$(3b + b_1)\frac{d}{12}$	$(3b + b_1)\frac{d^2}{12}$	$d\sqrt{\frac{3b + b_1}{6(b + b_1)}}$
$(b + 3b_1)\frac{d^3}{12}$	$(b + 3b_1)\frac{d^2}{12}$	$d\sqrt{\frac{b + 3b_1}{6(b + b_1)}}$
$\frac{\pi R^4}{4}$	$\frac{\pi R^3}{4}$	$\frac{R}{2}$
$\frac{\pi}{4}\left(R_e^4 - R_i^4\right)$	$\frac{\pi}{4}\left(\frac{R_e^4 - R_i^4}{R_e}\right)$	$\frac{\sqrt{R_e^2 + R_i^2}}{2}$
$0.1088\,R^4$	$0.2564\,R^3$	$0.2631\,R$
$0.1098(R_e^4 - R_i^4) - \frac{0.283\,R_e^2 R_i^2(R_e - R_i)}{R_e + R_i}$ $\doteqdot 0.3\,(R_e - R_i)\,R_o^3$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$

Section.	Area, A.	Distance of Neutral Axis from Extreme Fibres. y and y_1.
$s = \frac{d_e}{2}$ $d_i = .866\, d_e$  *Regular Hexagon.*	$\frac{3}{2} d_i^2 \tan 30° = 0.866\, d_i^2$ or $0.6495\, d_e^2$ or $2.5980\, S^2$	$\frac{d_i}{2}$ or $0.433\, d_e$
 *Regular Hexagon.*	$\frac{3}{2} d_i^2 \tan 30° = 0.866\, d_i^2$ or $0.6495\, d_e^2$ or $2.5980\, S^2$	$\frac{d_e}{2}$ or $0.577\, d_i$
 *Hexagonal Cell.*	$0.6495\,(d_1^2 - d_2^2)$	$\frac{\sqrt{3}}{4} d_1 = 0.433\, d_1$
$s = 0{\cdot}382\, d_e$, $d_i = {\cdot}9239\, d_e$  *Regular Octagon.*	$2\, d_i^2 \tan 22\frac{1}{2}° = 0.828\, d_i^2$ or $\frac{d_e^2}{\sqrt{2}} = 0.7071\, d_e^2$ or $4.828\, S^2$	$\frac{d_i}{2}$ or $\frac{d_e}{2} \cos 22\frac{1}{2}° = 0.462\, d_e$
 *Regular Octagon.*	$0.828\, d_i^2$ or $0.7071\, d_e^2$ or $4.828\, S^2$	$\frac{d_e}{2}$ or $\frac{d_i}{2 \cos 22\frac{1}{2}°} = 0.5412\, d_i$
 *Octagonal Cell.*	$0.707\,(d_1^2 - d_2^2)$	$\frac{d_1}{2} \cos 22\frac{1}{2}° = 0.462\, d_1$

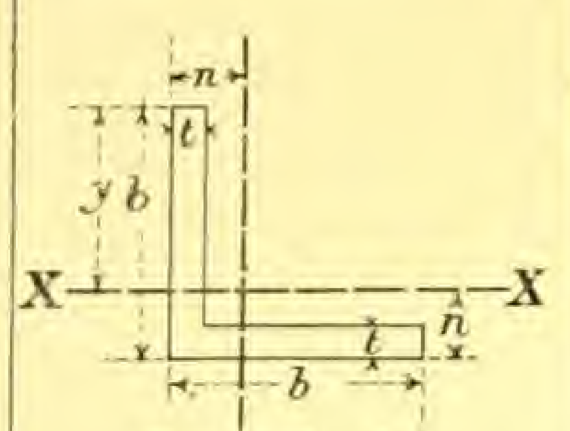
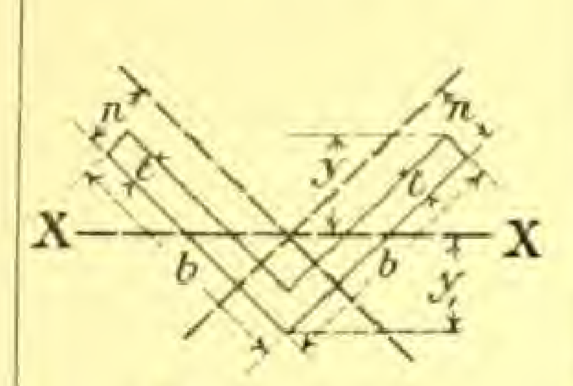
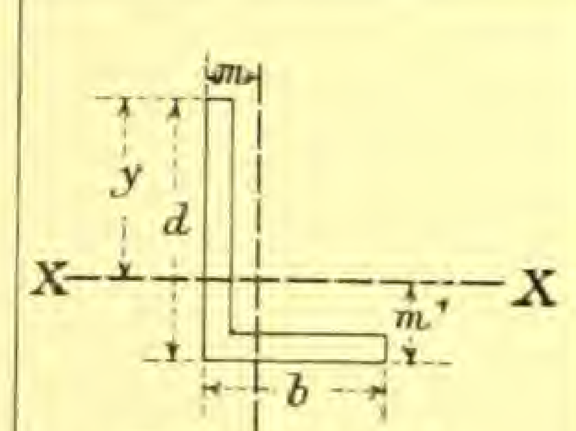
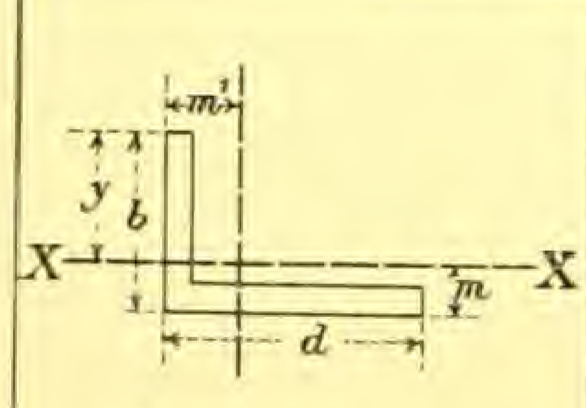
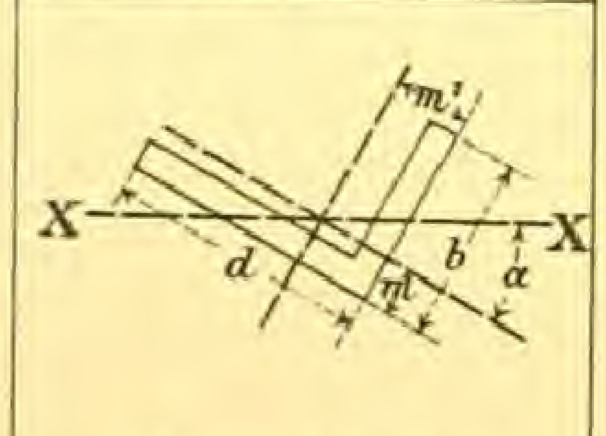
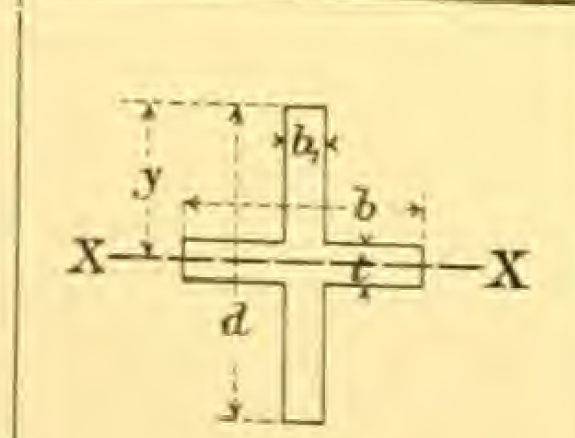
Moment of Inertia about xx. I_{xx}	Section Modulus. $z = \frac{I}{y}$.	Radius of Gyration. $r = \sqrt{\frac{I}{A}}$.
$0.06\, d_i^4$ or $0.5413\, S^4$ or $0.0338\, d_e^4$	$0.12\, d_i^3$ or $0.078\, d_e^3$	$0.2632\, d_i$ or $0.228\, d_e$
$0.06\, d_i^4$ or $0.5413\, S^4$ or $0.0338\, d_e^4$	$0.104\, d_i^3$ or $0.0676\, d_e^3$	$0.2632\, d_i$ or $0.228\, d_e$
$0.0338\,(d_1^4 - d_2^4)$	$0.078\,\frac{d_1^4 - d_2^4}{d_1}$	$0.2281\,\sqrt{\frac{d_1^4 - d_2^4}{d_1^2 - d_2^2}}$
$0.055\, d_i^4$ or $0.0399\, d_e^4$ or $1.88\, S^4$	$0.109\, d_i^3$ or $0.0864\, d_e^3$	$0.257\, d_i$ or $0.2376\, d_e$
$0.055\, d_i^4$ or $0.0399\, d_e^4$ or $1.88\, S^4$	$0.1016\, d_i^3$ or $0.0798\, d_e^3$	$0.257\, d_i$ or $0.2376\, d_e$
$0.1595\,(d_1^4 - d_2^4)$	$0.345\,\frac{d_1^4 - d_2^4}{d_1}$	$0.4749\,\sqrt{\frac{d_1^4 - d_2^4}{d_1^2 - d_2^2}}$

Section.	Area, A.	Distance of Neutral Axis from Extreme Fibres. y and y_1.
 Regular Hexagon.	$\frac{3}{2} d_i^2 \tan 30° = 0.866 d_i^2$ or $0.6495 d_e^2$ or $2.5980 S^2$	$\frac{d_i}{2}$ or $0.433 d_e$
 Regular Hexagon.	$\frac{3}{2} d_i^2 \tan 30° = 0.866 d_i^2$ or $0.6495 d_e^2$ or $2.5980 S^2$	$\frac{d_e}{2}$ or $0.577 d_i$
 Hexagonal Cell.	$0.6495 (d_1^2 - d_2^2)$	$\frac{\sqrt{3}}{4} d_1 = 0.433 d_1$
 Regular Octagon.	$2 d_i^2 \tan 22\frac{1}{2}° = 0.828 d_i^2$ or $\frac{d_e^2}{\sqrt{2}} = 0.7071 d_e^2$ or $4.828 S^2$	$\frac{d_i}{2}$ or $\frac{d_e}{2} \cos 22\frac{1}{2}° = 0.462 d_e$
 Regular Octagon.	$0.828 d_i^2$ or $0.7071 d_e^2$ or $4.828 S^2$	$\frac{d_e}{2}$ or $\frac{d_i}{2 \cos 22\frac{1}{2}°} = 0.5412 d_i$
 Octagonal Cell.	$0.707 (d_1^2 - d_2^2)$	$\frac{d_1}{2} \cos 22\frac{1}{2}° = 0.462 d_1$

Moment of Inertia about $x x$. I_{xx}	Section Modulus. $z = \frac{I}{y}$.	Radius of Gyration. $r = \sqrt{\frac{I}{A}}$.
$0.06\, d_i^4$ or $0.5413\, S^4$ or $0.0338\, d_e^4$	$0.12\, d_i^3$ or $0.078\, d_e^3$	$0.2632\, d_i$ or $0.228\, d_e$
$0.06\, d_i^4$ or $0.5413\, S^4$ or $0.0338\, d_e^4$	$0.104\, d_i^3$ or $0.0676\, d_e^3$	$0.2632\, d_i$ or $0.228\, d_e$
$0.0338\,(d_1^4 - d_2^4)$	$0.078\,\frac{d_1^4 - d_2^4}{d_1}$	$0.2281\sqrt{\frac{d_1^4 - d_2^4}{d_1^2 - d_2^2}}$
$0.055\, d_i^4$ or $0.0399\, d_e^4$ or $1.88\, S^4$	$0.109\, d_i^3$ or $0.0864\, d_e^3$	$0.257\, d_i$ or $0.2376\, d_e$
$0.055\, d_i^4$ or $0.0399\, d_e^4$ or $1.88\, S^4$	$0.1016\, d_i^3$ or $0.0798\, d_e^3$	$0.257\, d_i$ or $0.2376\, d_e$
$0.1595\,(d_1^4 - d_2^4)$	$0.345\,\frac{d_1^4 - d_2^4}{d_1}$	$0.4749\sqrt{\frac{d_1^4 - d_2^4}{d_1^2 - d_2^2}}$

Section.	Area, A.	Distance of Neutral Axis from Extreme Fibres. y and y_1.
 *Ellipse.*	$\frac{\pi}{4} b d$	$\frac{d}{2}$
 *Ellipse.*	$\frac{\pi}{4} b d$	$\frac{b}{2}$
 *Hollow Ellipse.*	$\frac{\pi}{4}(b d - b_1 d_1)$	$\frac{d}{2}$
 *Parabola.*	$\frac{2}{3} b d$	$y = \frac{3}{5} d$ $y_1 = \frac{2}{5} d$
 *Parabola.*	$\frac{2}{3} b d$	d
 *Parabola.*	$\frac{2}{3} b d$	$\frac{b}{2}$

Moment of Inertia about $x\,x$. I_{xx}.	Section Modulus. $z = \dfrac{I}{y}$.	Radius of Gyration. $r = \sqrt{\dfrac{I}{A}}$.
$\dfrac{\pi\, b\, d^3}{64}$	$\dfrac{\pi\, b\, d^2}{32}$	$\dfrac{d}{4}$
$\dfrac{\pi\, d\, b^3}{64}$	$\dfrac{\pi\, d\, b^2}{32}$	$\dfrac{b}{4}$
$\dfrac{\pi}{64}\left(b\,d^3 - b_1\,d_1^3\right)$	$\dfrac{\pi}{32\,d}\left(b\,d^3 - b_1\,d_1^3\right)$	$\frac{1}{4}\sqrt{\dfrac{b\,d^3 - b_1\,d_1^3}{b\,d - b_1\,d_1}}$
$\dfrac{8}{175}\,b\,d^3 = 0.0457\;b\,d^3$	$\dfrac{8}{105}\,b\,d^2 = 0.0762\;b\,d^2$	$0.262\,d$
$\dfrac{16}{105}\,b\,d^3 = 0.1524\;b\,d^3$	$\dfrac{16}{105}\,b\,d^2 = 0.1524\,b\,d^2$	$\sqrt{\dfrac{8}{35}}\,d = 0.478\,d$
$\dfrac{d\,b^3}{30}$	$\dfrac{d\,b^2}{15}$	$\dfrac{b}{\sqrt{20}} = 0.2236\,b$

Section.	Area, A	Distance of Neutral Axis from Extreme Fibres. y and y_1.
 *Equal Angle*	$t(2b - t)$	$y_1 = n = \dfrac{b^2 + bt - t^2}{2(2b - t)}$ $\doteq 0.31\,b$
 *Equal Angle.*	$t(2b - t)$	$y_1 = \sqrt{2}\,n = 1.414\,n$ $y = 0.707\,\{(b + t) - 2n\}$
 *Unequal Angle.*	$t(b + d - t)$	$y_1 = m^1 = \dfrac{t(2d_1 + b) + d_1^2}{2(d_1 + b)}$
 *Unequal Angle.*	$t(b + d - t)$	$y_1 = m = \dfrac{t(2b_1 + d) + b_1^2}{2(b_1 + d)}$
 *Unequal Angle.*	$t(b + d - t)$	—
 *Cruciform.*	$bd - 2(b - b_1)(d - t)$	$\dfrac{d}{2}$

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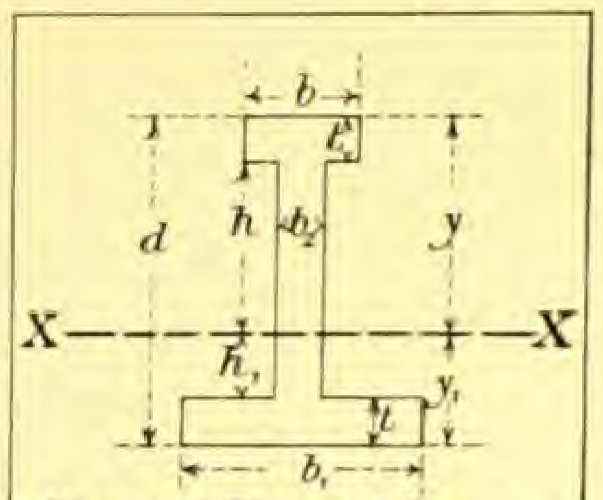
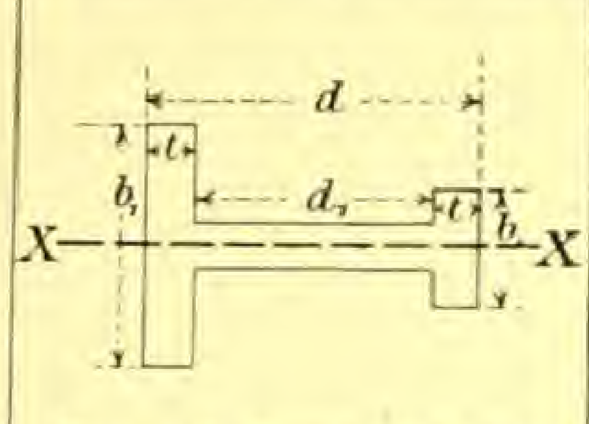
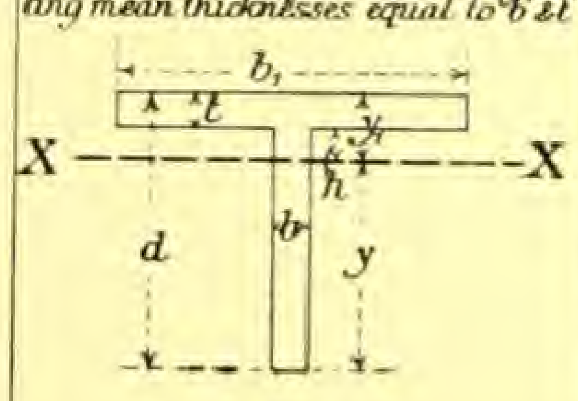
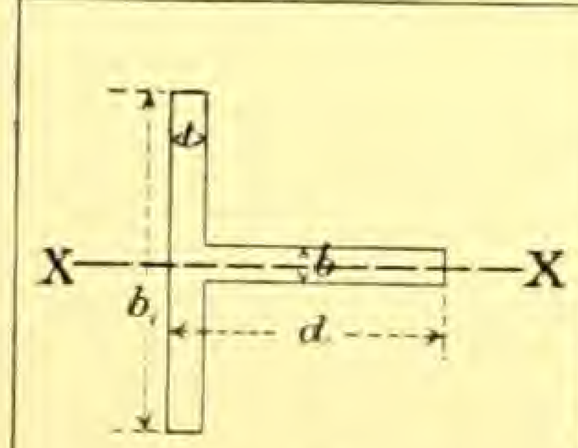
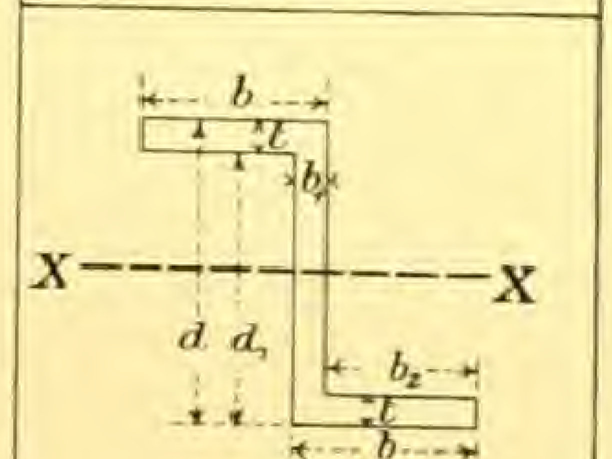
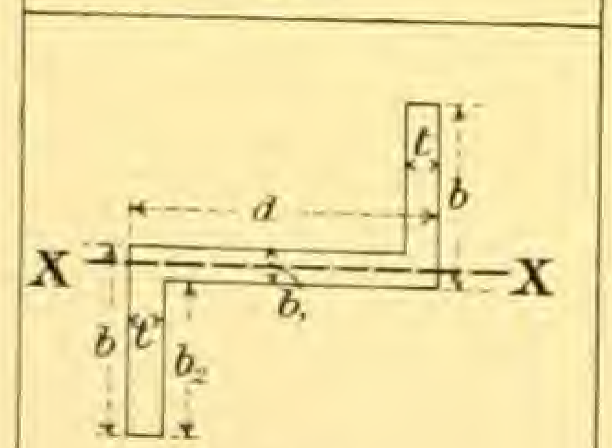
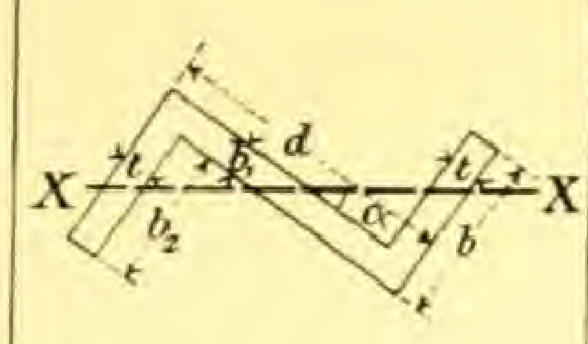
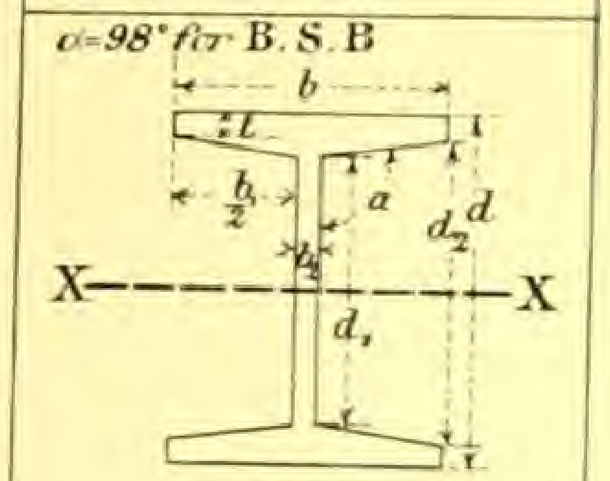
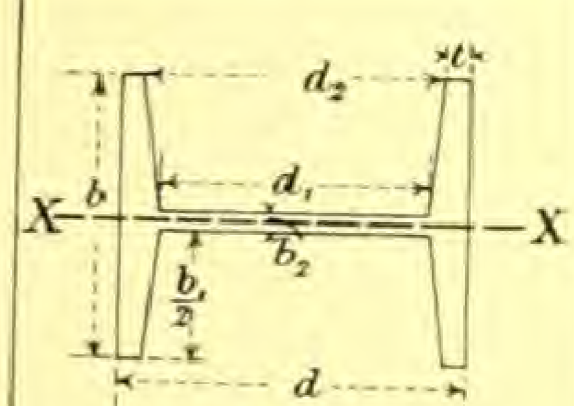
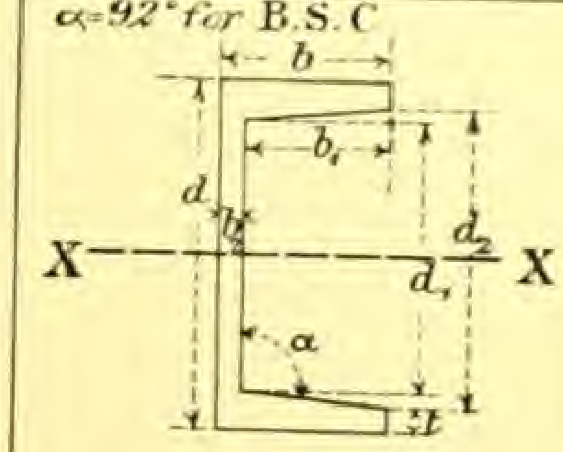
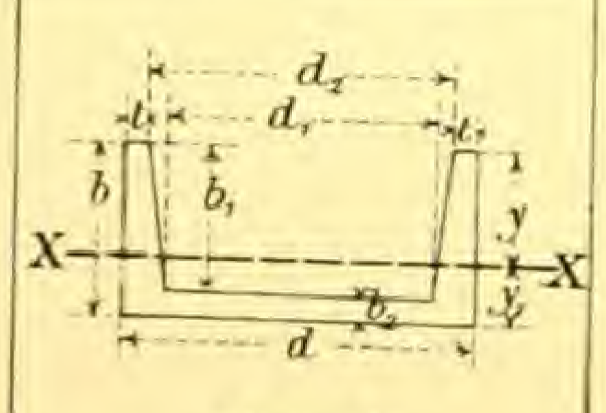
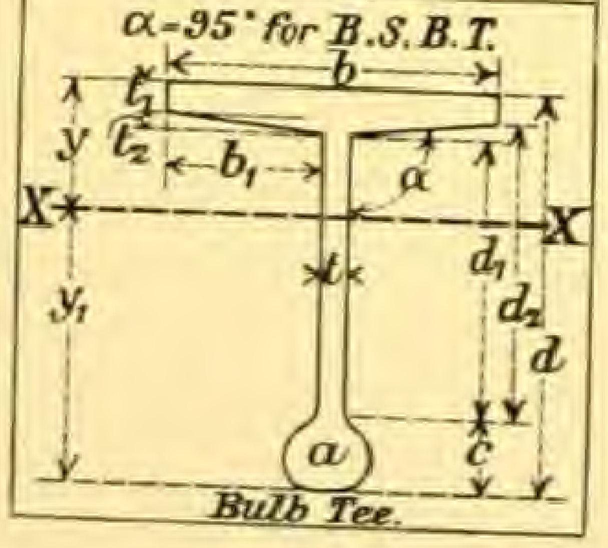
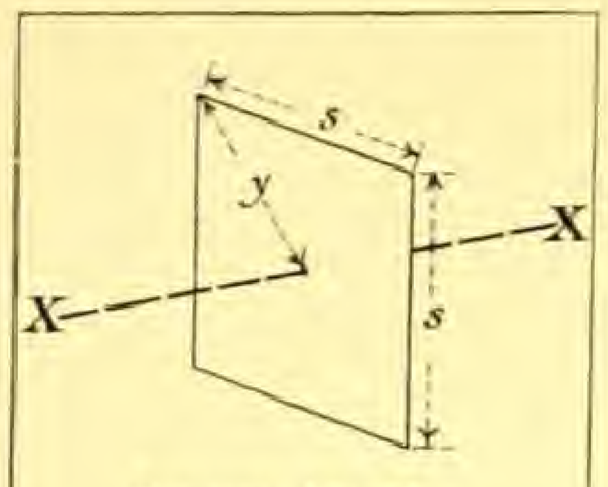
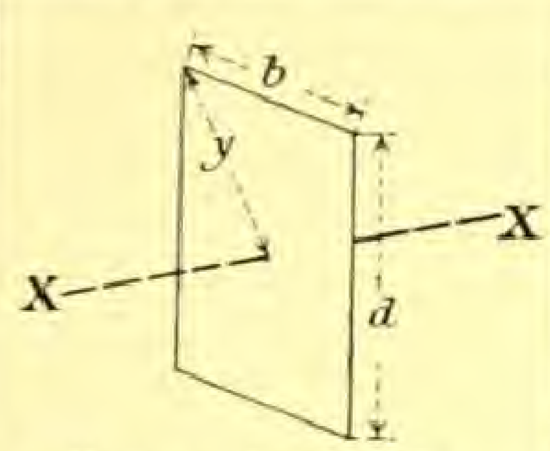
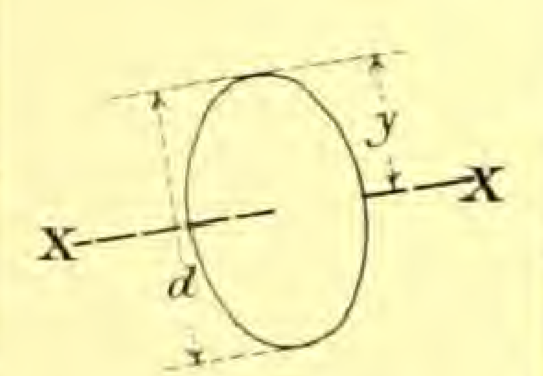
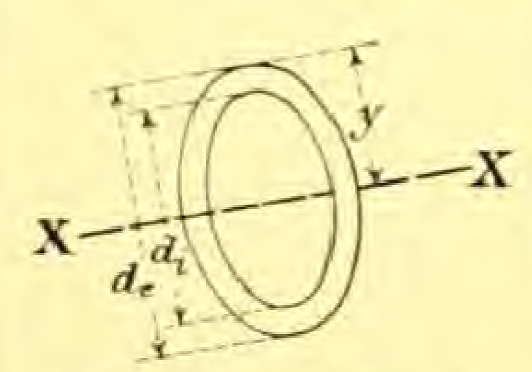
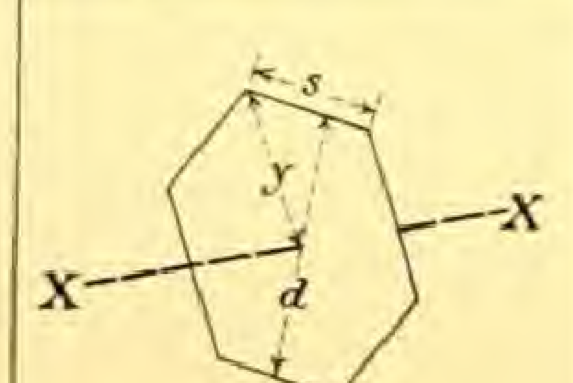
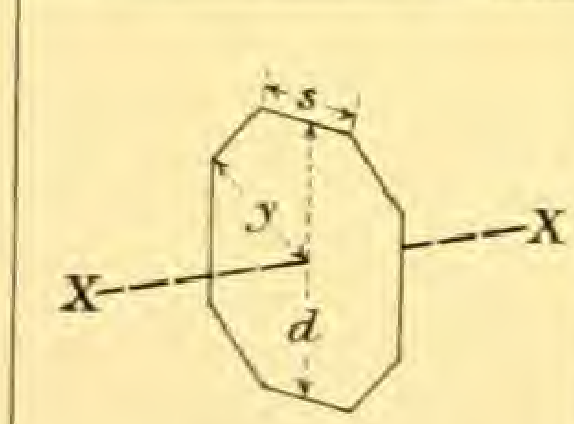
Moment of Inertia about $x\,x$. I_{xx}	Section Modulus. $z = \frac{\mathrm{I}}{y}$.	Radius of Gyration. $r = \sqrt{\frac{\mathrm{I}}{\mathrm{A}}}$.
$\frac{t(b-n)^3 + b\,n^3 - (b-t)(n-t)^3}{3}$	$\frac{\mathrm{I}}{y}$	$\sqrt{\frac{\mathrm{I}}{\mathrm{A}}}$
$\frac{2\,n^4 - 2\,(n-t)^4 + t\left\{b - \left(2\,n - \frac{t}{2}\right)\right\}^3}{3}$	$\frac{\mathrm{I}}{y}$	$\sqrt{\frac{\mathrm{I}}{\mathrm{A}}} \doteqdot \sqrt{\frac{b^2}{24}}$
$\mathrm{I}_1 = \frac{t(d-m^1)^3 + b\,(m^1)^3 - (b-t)(m^1-t)^3}{3}$	$\frac{\mathrm{I}}{y}$	$\sqrt{\frac{\mathrm{I}}{\mathrm{A}}}$
$\mathrm{I}_2 = \frac{t(b-m)^3 + d\,m^3 - (d-t)(m-t)^3}{3}$	$\frac{\mathrm{I}}{y}$	$\sqrt{\frac{\mathrm{I}}{\mathrm{A}}}$
$\mathrm{I}_3 = \frac{\mathrm{I}_2 \cos^2 \alpha - \mathrm{I}_1 \sin^2 \alpha}{\cos 2\,\alpha}$ $\alpha = \frac{1}{2} \tan^{-1}\left[\frac{d\,(2\,m - t)(d - 2\,m^1)}{2\,(\mathrm{I}_1 - \mathrm{I}_2)} + \right.$ $\left. + \frac{(2\,m^1 - t)(b-t)(b + t - 2\,m)}{2\,(\mathrm{I}_1 - \mathrm{I}_2)}\right]t$	$\frac{\mathrm{I}}{y}$	$\sqrt{\frac{\mathrm{I}}{\mathrm{A}}} \doteqdot \sqrt{\frac{b^2 d^2}{12\,(b^2 + d^2)}}$
$\frac{b_1 d^3 + t^3 (b - b_1)}{12}$	$\frac{b_1 d^3 + t^3 (b - b_1)}{6\,d}$	$\sqrt{\frac{b_1 d^3 + t^3 (b - b_1)}{12\,\{b\,d - (b - b_1)(d - t)\}}}$

Section	Area, A.	Distance of Neutral Axis from Extreme Fibres. y and y_1.
 Unequal Flanged Beam.	$b\,t + b_1\,t + b_2\,(d - 2\,t)$	$y_1 = \dfrac{b_2\,d^2 + (b_1 - b_2)\,t^2 + t\,(b - b_2)\,(2\,d - t)}{2\,A}$
 Unequal Flanged Beam.	$b\,t + b_1\,t + b_2\,(d - 2\,t)$	$\dfrac{b_1}{2}$ $\dfrac{b}{2}$
NOTE. In case of Taper Tee results correct to about 1% may be obtained by putting mean thicknesses equal to b & t.  Tee.	$b_1\,t + b\,(d - t)$	$y_1 = \dfrac{b\,d^2 + (b_1 - b)\,t^2}{2\{b_1\,t + b\,(d - t)\}} \doteq 0.3\,d$
 Tee.	$b_1\,t + b\,(d - t)$	$\dfrac{b_1}{2}$
 Zed.	$2\,b\,t + b_1\,(d - 2\,t)$	$\dfrac{d}{2}$
 Zed.	$2\,b\,t + b_1\,(d - 2\,t)$	$b - \dfrac{b_1}{2}$

Moment of Inertia about $x x$. I_{xx}	Section Modulus. $z = \frac{I}{y}$.	Radius of Gyration. $r = \sqrt{\frac{I}{A}}$.
$\frac{b_1(y_1^3 - h_1^3) + b(y^3 - h^3) + b_2(h^3 + h_1^3)}{3}$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
$\frac{(b + b_1)t^3 + d_1 b_2^3}{12}$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
$\frac{b_1 y_1^3 + b y^3 - (b_1 - b) h^3}{3}$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
$\frac{t b_1^3 + (d - t) b^3}{12}$	$\frac{I}{y} \doteqdot \frac{t b_1^2}{6}$	$\sqrt{\frac{I}{A}}$
$\frac{b d^3 - b_2 (d - 2t)^3}{12} = I_1$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
$\frac{d(b + b_2)^3 - 2 d_1 b_2^3 - 6 b_2 d_1 b^2}{12} = I_2$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$

Section.	Area, A.	Distance of Neutral Axis from Extreme Fibres. y and y_1.
 *Zed.*	$2\,b\,t + b_1\,(d - 2\,t)$	—
$\alpha = 98°$ for B.S.B  *Beam.*	$b_2 d_2 + 2\,b\,t + \frac{b_1}{2}\left(d_2 - d_1\right)$ $= b_2 d_2 + 2\,b\,t$ when flanges are parallel	$\frac{d}{2}$
 *Beam.*	$b_2 d_2 + 2\,b\,t + \frac{b_1}{2}\left(d_2 - d_1\right)$ $= b_2 d_2 + 2\,b\,t$ when flanges are parallel	$\frac{b}{2}$
$\alpha = 92°$ for B.S.C  *Channel.*	$b_2 d_2 + 2\,b\,t + \frac{b_1}{2}\left(d_2 - d_1\right)$ $= b_2 d_2 + 2\,b\,t$ when flanges are parallel	$\frac{d}{2}$
 *Channel.*	$b_2 d_2 + 2\,b\,t + \frac{b_1}{2}\left(d_2 - d_1\right)$ $= b_2 d_2 + 2\,b\,t$ when flanges are parallel	$\frac{d_2 - d_1}{2} = h$ $y_1 = \frac{1}{6\,A}\left[3\,d\,b^2 - 6\,b_1 b_2\,(d_1 + h) - b_1^2\,(3\,d_1 + 4\,h)\right]$ $= \frac{1}{2\,A}\left[d\,b_2 - 2\,b_1\,b_2\,d_1 - d_1\,b_1^2\right]$ when flanges are parallel
$\alpha = 95°$ for B.S.B.T.  *Bulb Tee.*	Let a = area of head $a + t\,(d - c) + 2\,b_1\left(t_1 + \frac{t_2}{2}\right)$	$y = \frac{1}{2\,A}\left[a\,(2\,d - c) + t\,(d - c)^2 + 2\,b_1\left(t_1 + \frac{t_2}{2}\right)^2\right]$

Moment of Inertia about $x x$. I_{xx}	Section Modulus. $z = \frac{I}{y}$.	Radius of Gyration. $r = \sqrt{\frac{I}{A}}$.
$\frac{I_2 \cos^2 \alpha - I_1 \sin^2 \alpha}{\cos 2\alpha}$ where $\alpha = \frac{1}{2}\tan^{-1}\left[-\frac{(dt - t^2)(b^2 - bt)}{I_1 - I_2}\right]$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
S = slope of flange = $\frac{d_2 - d_1}{b_1}$ $\frac{1}{12}\left[b\,d^3 - \frac{1}{4\,S}\left(d_2^4 - d_1^4\right)\right]$ $= \frac{1}{12}\left[b\,d^3 - b_1\,d_1^3\right]$ when flanges are parallel	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
$\frac{1}{12}\left[2\,t\,b^3 + d_1\,b_2^3 + \frac{S}{4}\left(b^4 - b_2^4\right)\right]$ $= \frac{1}{12}\left[2\,t\,b^3 + d_1\,b_2^3\right]$ when flanges are parallel	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
S = slope of flange = $\frac{d_2 - d_1}{2\,b_1}$ $\frac{1}{12}\left[b\,d^3 - \frac{1}{8\,S}\left(d_2^4 - d_1^4\right)\right]$ $= \frac{1}{12}\left[b\,d^3 - b_1\,d_1^3\right]$ when flanges are parallel	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
$\frac{1}{3}\left[2tb^3 + d_1\,b_2^3 + \frac{S}{2}\left(b^4 - b_2^4\right)\right] - A\,y_1^2$ $= \frac{1}{3}\left[d\,(y^3 + y_1^3) - d_1\,\{y^3 + (y_1 - b_2)^3\}\right]$ when flanges are parallel	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
$\frac{a}{16}\left[c^2 + 4\,(2\,d_2 + 3\,c)^2\right] + \frac{td_2^3}{3} + \frac{b_1t_2^3 + 2\,bt_1^3}{6} - A\,(y - t_1)^2$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$

Section.	Area, A.	Distance of Neutral Axis from Extreme Fibres. y and y_1.
 *Square Shaft.*	S^2	$\frac{S\sqrt{2}}{2} = 0.707\,S$
 *Rectangular Shaft.*	$b\,d$	$\frac{\sqrt{b^2 + d^2}}{2}$
 *Round Shaft.*	$\frac{\pi d}{4}$	$\frac{d}{2}$
 *Hollow Round Shaft.*	$\frac{\pi}{4}\left(d_e^2 - d_i^2\right)$	$\frac{d_e}{2}$
 *Hexagonal Shaft.*	$\frac{3}{2}\,d^2 \tan 30^\circ = 0.866\,d^2$ or $2.598\,S^2$	$0.577\,d$ or S
 *Octagonal Shaft.*	$2\,d^2 \tan 22\frac{1}{2}^\circ = 0.828\,d^2$ or $4.828\,S^2$	$0.5412\,d$ or $1.309\,S$

Moment of Inertia about $x\,x$. I_{xx}	Section Modulus. $z = \frac{I}{y}$.	Radius of Gyration. $r = \sqrt{\frac{I}{A}}$.
$\frac{S^4}{6}$	$\frac{S^3}{3\sqrt{2}} = 0.2357\,S^3$ $z = 0.208\,S^3$ (St. Venant)	$\frac{S}{\sqrt{6}} = 0.4083\,S$
$\frac{b\,d\,(b^2 + d^2)}{12}$	$\frac{b\,d\,\sqrt{b^2 + d^2}}{6}$ $z = 0.2944\ \frac{b^2 d^2}{\sqrt{b^2 + d^2}}$ (St. Venant)	$\sqrt{\frac{b^2 + d^2}{12}} = 0.2887\ \sqrt{b^2 + d^2}$
$\frac{\pi\,d^4}{32}$	$\frac{\pi\,d^3}{16}$	$\frac{d}{\sqrt{8}} = 0.3536\,d$
$\frac{\pi\,(d_e^4 - d_i^4)}{32}$	$\frac{\pi}{16}\ \frac{d_e^4 - d_i^4}{d_e}$	$\sqrt{\frac{d_e^4 - d_i^4}{8\,(d_e^2 - d_i^2)}}$
$0.12\,d^4$ or $1.0826\,S^4$	$0.208\,d^3$ or $1.0826\,S^3$	$0.3722\,d$ or $0.6455\,S$
$0.11\,d^4$ or $3.76\,S^4$	$0.2032\,d$ or $2.872\,S$	$0.3644\,d$ or $0.8824\,S$

Beams of Solid Cross Section.

THE ordinary formulæ for ascertaining the stress on the extreme fibres of a beam do not give true results for beams of solid cross section. The maximum difference occurs with beams of solid section, and gradually decreases as the section more nearly approaches that of an ordinary built-up girder, when it is so small that it can generally be neglected in practice. In the case of a rectangular beam of cast iron, for instance, the load that will actually break it is about $2\frac{1}{4}$ times the load that the ordinary formula shows would cause a stress on the extreme fibres, equal to the ultimate strength of the material, as given by direct tensile tests. The following approximate method of calculating the transverse strength of the class of beams above referred to will be found to give results sufficiently accurate for most purposes.

Let f = The ultimate resistance to direct tension.

F = The apparent resistance to the same force caused by transverse stress.

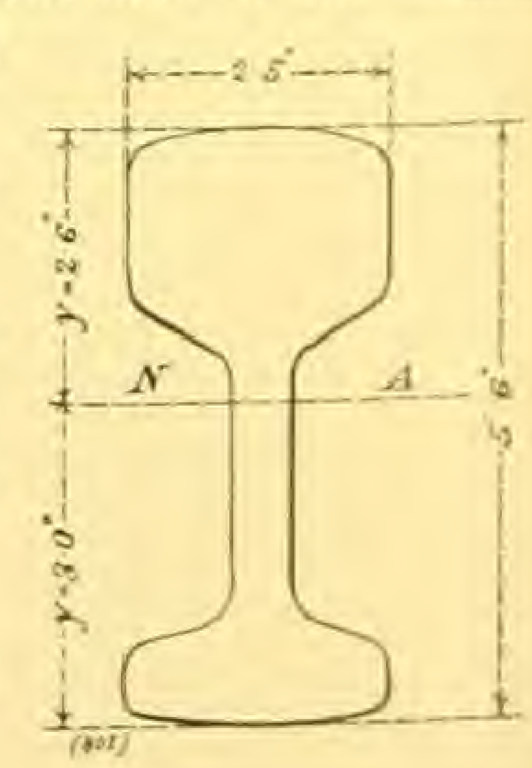
Q = A constant multiplied by f.

Then $F = f + Q$.

The value of Q for beams of different cross section is as follows :—

Section.	Cast Iron.	Wrought Iron.	Steel.
Rectangular ... ... Q =	$1\frac{1}{4}f$	$\frac{9}{16}f$	$\frac{11}{16}f$
Round ... ... Q =	$1\frac{1}{2}f$	$\frac{11}{16}f$	$\frac{13}{16}f$
Square (in direction of diagonal) Q =	$1\frac{1}{4}f$	$\frac{13}{16}f$	$\frac{7}{8}f$

For beams of cross section varying between a solid rectangle and an ordinary built-up girder, such as a rail, for example, the value of Q can be obtained with sufficient accuracy by multiplying its value for the rectangular section by the area of the section under consideration, and dividing by the area of the enclosing rectangle.



The following example shows the application of the above method to a bull-headed rail :—

The average of four tests, of a bull-headed rail, 5.6 in. deep by 2.5 in. wide, weighing $82\frac{1}{2}$ lb. per yard, tested as a beam of 60 in. span, gave the central load causing failure to be 35 tons, required the actual tensile stress obtained.

The position of the neutral axis will be found to be 2.6 in. from the head and 3.0 in. from the bottom flange of the rail, and the moment of inertia 28.5 in.^{4}

For a load of 35 tons at the centre of the 60 in. span, $M = \frac{W\,L}{4} = 525$ inch tons,

and as $M = F\,\frac{I}{y}$,

$$\therefore F = \frac{525 \times 3}{28.5} = 55.3 \text{ tons per square inch.}$$

The value of Q in this case is $\frac{11}{16} f \times \frac{\text{area of rail}}{\text{area of enclosing rectangle.}}$

$$\therefore Q = \frac{11}{16} f \times \frac{8.05}{5.6 \text{ in.} \times 2.5 \text{ in.}} = 0.4\,f.$$

And as $F = f + Q$ substituting value of Q.

$$F = f + 0.4\,f = 55.3 \text{ tons per square inch.}$$

And therefore actual tensile stress $f = 39.5$ tons per square inch.

The strength of strips cut out of the bottom flange of the rail, and tested in direct tension was 39 tons per square inch.[1]

Deflection of Solid Beams.

The anomalies presented by beams of different cross section, as regards strength, do not extend to their deflections, except that the elastic range is increased, and consequently the maximum deflection within the elastic limit is greater than theory would indicate.

[1] "Minutes of Proceedings of the Institution of Civil Engineers," vol. xlvi, p. 188.

Deflection of Framed Structures.

THE deflection of a framed structure, due to a given load, can be found either analytically or graphically, when the elastic deformation of each member, caused by the stress upon it, is known, as shown by the following examples.

CASE I.—*Required the movement of the point* B *of the frame* A B C, *due to the weight* W.

Let l = Length of any member in inches, and L = its length in feet.

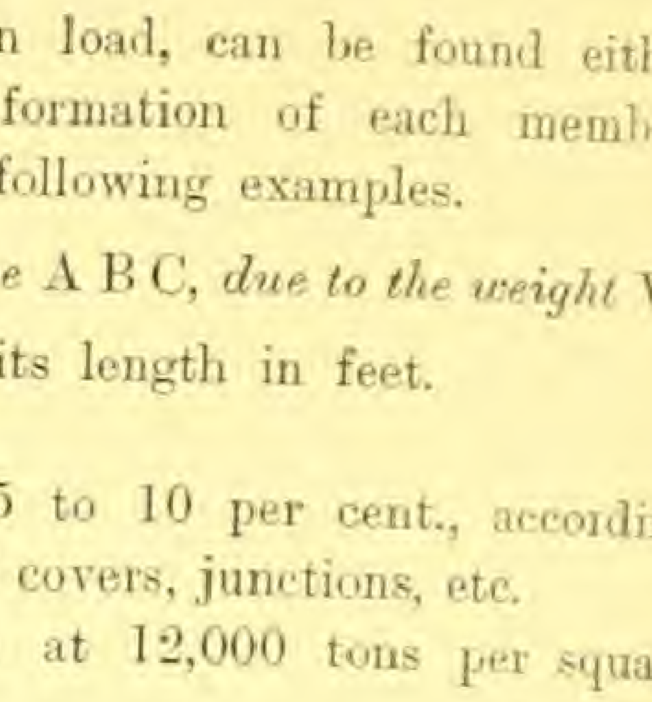
S = Total stress on the member.

A = Gross area of member in square inches + 5 to 10 per cent., according to the design, to allow for the effect of covers, junctions, etc.

E = Modulus of elasticity, which may be taken at 12,000 tons per square inch, if W is measured in tons.

Then the elastic deformation of a member = $\frac{l\,S}{E\,A}$ or $\frac{L\,S}{1000\,A}$ if L is taken in feet.

Suppose A B = 40 ft. and B C = 50 ft., and let the gross sectional area of each member be 30 and 60 square inches respectively. If W = 100 tons then the stress on A B is 133 tons, and that on B C = 166 tons.



By the above formulæ the deformation or lengthening of A B will be 0.178 in., and the shortening of B C will be 0.139 in.

With any convenient scale make B c and B d = to the deformation of A B and B C, and draw the perpendiculars c E and d E, then their point of intersection E, is the new position of B, and by measuring c E by the same scale, the deflection of B = 0.47 in. is obtained.

The above manner of determining deflections has been fully described and applied to many forms of braced structures by Allan D. Stewart.[1] It is not, however, so convenient as the analytical method, first suggested by Lamé and afterwards developed by Maxwell, Winkler and Mohr. The following examples show this method applied to such cases as generally occur in actual practice. Considering first the case already taken, we have:—

CASE I.—*Required the movement of the point* B *of the frame* A B C *due to the weight* W.

Let l = Length of any member in inches, and L = its length in feet.

S = Total stress on a member due to W = 100 tons.

P = Stress per square inch on a member due to W.

u = Total stress on a member due to a weight of 1 ton placed at the point, the deflection of which it is desired to ascertain.

A = Gross area of member in square inches + 5 to 10 per cent., according to the design, to allow for the effect of covers, junctions, etc.

E = Modulus of elasticity, which may be taken at 12,000 tons per square inch, if W is measured in tons.

[1] "Minutes of Proceedings of the Institution of Civil Engineers, vol. cix., page 269.

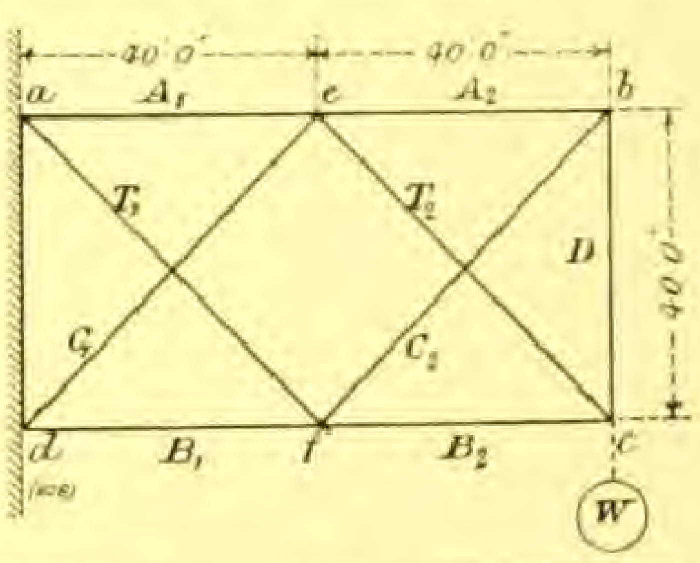
Then the elastic deformation of a member $= \frac{l\,S}{E\,A} = \frac{L\,S}{1000\,A}$ if L is taken in feet.

The stresses, areas, deformation, etc., of the members under consideration will be as shown in the following Table :—

Member.	L Length.	S Total Stress.	A Area.	P Stress per Square Inch.	$\frac{P\,l}{E.}$	$u.$	Contribution to Deflection $= \frac{P\,l\,u}{E.}$	Summation.
	ft.	tons	sq. in.	tons	in.	tons	in.	in.
A B	40	− 133	30	− 4.44	− 0.178	− 1.33	+ 0.237	
B C	50	+ 166	60	+ 2.77	+ 0.139	+ 1.66	+ 0.230	
							Total movement =	0.467

The last column gives the deflection of the point B = 0.467 in., which is the same result as already obtained by the graphical method.

CASE II.—*Required, the movement of the point c of a cantilever a b c d due to the weight* W.



Let the length of the cantilever be 80 ft., the depth 40 ft., and W = 200 tons.

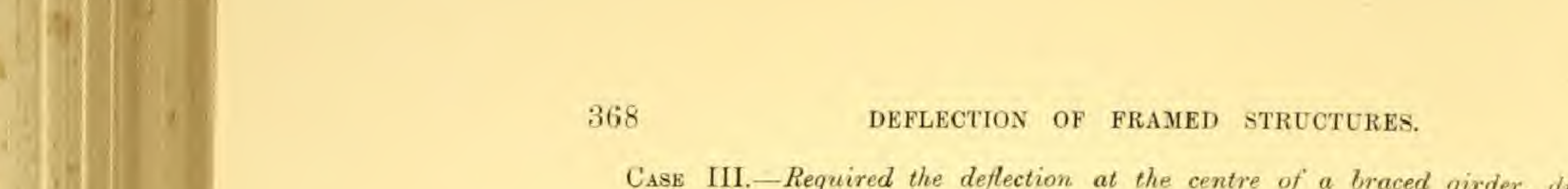
The stresses may be calculated on the assumption that the bar D transfers one-half of the load to b, although this is not strictly correct. Having fixed the limiting stress per square inch for each member, their areas can be obtained, and the following Table prepared as for the previous example.

Member.	L Length.	S Total Stress.	A Area.	P Stress per Square Inch.	$\frac{P\,l}{E.}$	$u.$	Contribution to Deflection $= \frac{P\,l\,u}{E.}$	Summation.
	ft.	tons	sq. in.	tons	in.	tons	in.	in.
A_1	40	− 300	60	− 5.0	− 0.20	− 1.5	+ 0.30	
A_2	40	− 100	30	− 3.3	− 0.13	− 0.5	+ 0.06	
B_1	40	+ 300	60	+ 5.0	+ 0.20	+ 1.5	+ 0.30	
B_2	40	+ 100	30	+ 3.3	+ 0.13	+ 0.5	+ 0.06	+ 0.72
T_1	$56\frac{1}{2}$	− 141	30	− 4.7	− 0.27	− 0.7	+ 0.19	
T_2	$56\frac{1}{2}$	− 141	30	− 4.7	− 0.27	− 0.7	+ 0.19	
C_1	$56\frac{1}{2}$	+ 141	50	+ 2.8	+ 0.16	+ 0.7	+ 0.11	
C_2	$56\frac{1}{2}$	+ 141	50	+ 2.8	+ 0.16	+ 0.7	+ 0.11	+ 0.60
D	40	− 100	20	− 5.0	− 0.20	− 0.5	+ 0.10	
							Total deflection ...	1.32

The required movement, therefore, of the point c is 1.32 in., the member D not contributing to the deflection.

The last column in the above Table shows also that the flange contributes to the deflection 0.72 in., and the web 0.60 in., and dispels the general opinion that the deflection of a lattice girder is but little affected by the web.

CASE III.—*Required the deflection at the centre of a braced girder, due to a distributed load.*



Let the span of the girder be 100 ft., its depth 10 ft., and the distributed load 100 tons.

The stresses due to the distributed load of 100 tons must first be calculated, and the sectional area of all the members obtained. The stresses due to 1 ton at the centre of the girder, the point at which the deflection is required, must next be ascertained, to allow the following table to be prepared.

Member.	L Length	S Total Stress.	A Area.	P Stress per Square Inch.	$\frac{Pl}{E}$.	u	Contribution to Deflection $\frac{Plu}{E}$.	Summation.
Top Boom	ft.	tons	sq. in.	tons	in.	tons	in.	in.
Bay 1	10	+ 45	9	+ 5	+ 0.05	+ 0.5	+ 0.025	
„ 2	10	+ 80	16	+ 5	+ 0.05	+ 1.0	+ 0.050	
„ 3	10	+ 105	21	+ 5	+ 0.05	+ 1.5	+ 0.075	
„ 4	10	+ 120	24	+ 5	+ 0.05	+ 2.0	+ 0.100	
„ 5	10	+ 125	25	+ 5	+ 0.05	+ 2.5	+ 0.125	
								+ 0.375
Bottom Boom								
Bay 1	10	Nil	Nil	Nil	Nil	Nil.	Nil.	
„ 2	10	− 45	7.5	− 6	− 0.06	− 0.5	+ 0.03	
„ 3	10	− 80	13.4	− 6	− 0.06	− 1.0	+ 0.06	
„ 4	10	− 105	17.5	− 6	− 0.06	− 1.5	+ 0.09	
„ 5	10	− 120	20.0	− 6	− 0.06	− 2.0	+ 0.12	
								+ 0.300
Web.								
Tie 1	14.14	− 65	13	− 5	− 0.0707	− 0.707	+ 0.05	
„ 2	14.14	− 50	10	− 5	− 0.0707	− 0.707	+ 0.05	
„ 3	14.14	− 35	7	− 5	− 0.0707	− 0.707	+ 0.05	
„ 4	14.14	− 21	4.5	− 5	− 0.0707	− 0.707	+ 0.05	
„ 5	14.14	− 7	1.4	− 5	− 0.0707	− 0.707	+ 0.05	
								+ 0.250
End Post	10	+ 45	11.25	+ 4	+ 0.04	+ 0.5	+ 0.02	
Strut 1	10	+ 35	8.75	+ 4	+ 0.04	+ 0.5	+ 0.02	
„ 2	10	+ 25	6.25	+ 4	+ 0.04	+ 0.5	+ 0.02	
„ 3	10	+ 15	3.75	+ 4	+ 0.04	+ 0.5	+ 0.02	
„ 4	10	+ 5	1.25	+ 4	+ 0.04	+ 0.5	+ 0.02	+ 0.100
							Half sum	+ 1.025 2
							Total deflection	+ 2.050

The required deflection of the girder is thus found to be 2.05 in., the flanges contributing 1.35 in., and the web 0.70 in.

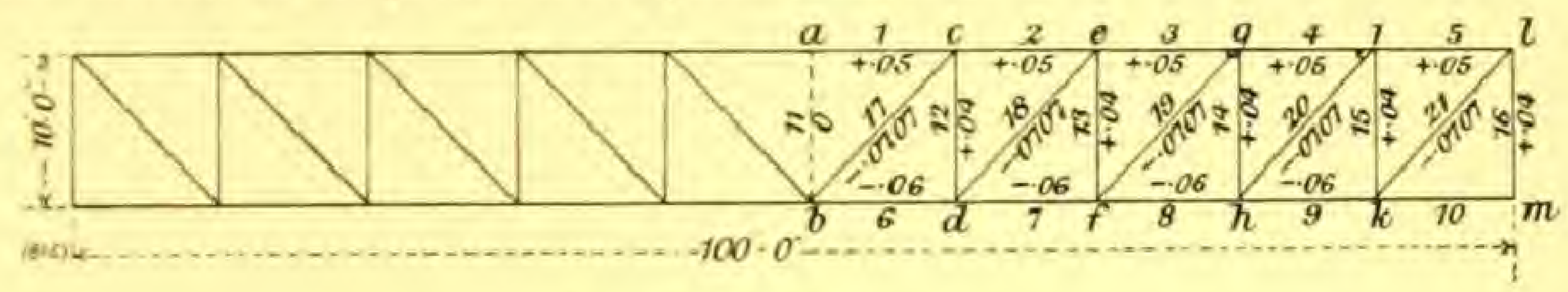
It should be remembered that the load of one ton, for finding the value of u for each

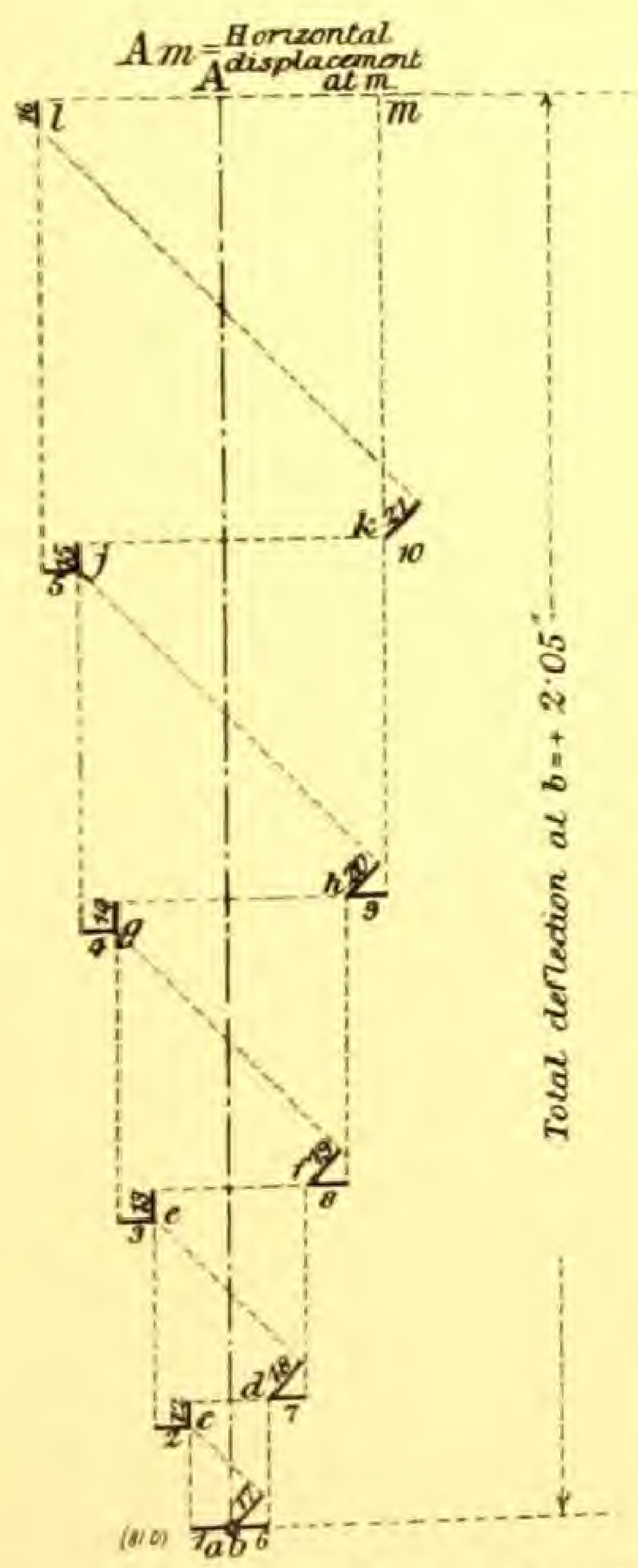
member, must always be placed at the point of the truss, the movement of which is to be ascertained, and in the direction in which the movement is to be measured.

This method of calculating deflections is fully described in "Modern Framed Structures," by Johnson, Bryan and Turneaure.

CASE III (a).—*Graphical method.*

The deflection at any panel point of the braced girder in Case III. may also be found by a graphical construction, when the Tables on the preceding page up to $\frac{P\,l}{E}$ have been obtained. Consider one-half of the girder only, as it is symmetrical, and letter it as shown. Assume that point a is fixed in position, that line $a\,b$ is fixed in direction, and that the motion of all points takes place relative to a.





DISPLACEMENT DIAGRAM.

Take any pole a. (In this case a and b are the same point.) Apply the principle explained under Case I., plotting elastic deformations $\frac{P\,l}{E}$, having regard to sign, and treating each panel point in succession. The deflection diagram is then obtained. The vertical distance from a to any panel point measures the total vertical displacement at that

panel point. The horizontal distance from the vertical through pole a to any panel point measures the total horizontal displacement at that point. Thus a A = 2.05 in. is the total vertical deflection at a, and A m is the horizontal movement of m relative to a or b.

Girders with Plate Webs.

The analytical or graphical method described may be applied to determine approximately the deflection by substituting diagonals between the vertical web stiffeners and applying the following empirical rule :—Assume a stress per square inch in the substituted vertical and diagonal members which is the same as that due to the vertical shearing stress on the web plate at the points of substitution.

Riveted Girders: Actual and Theoretical Deflection.

The difference between the actual and calculated values is due to various causes: the stiffness of riveted joints and cover plates; the quality of the workmanship; and the position and form of the floor, in the case of bridges, tend to reduce the deflection. It is found that the riveted web connections and joint cover plates reduce the deflection from 20 to 30 per cent. Allowance must also be made for the additional resistance of the material of the floor.

Deflection Due to Temperature.

The analytical or graphical method may be applied to determine the deformation of a girder or frame due to a rise or fall of temperature, acting on the girder as a whole, or on one flange only. Expansions take the minus sign like tensile strains. Contractions take the plus sign like compressive strains. The total deformation of any member is then $\delta_T = \dfrac{P\,l\,u}{E} + \alpha\,t\,l$, the temperature effect being expressed by the second term where α = thermal linear coefficient of expansion of the material of the member, or increase of length per unit length per degree rise of temperature Fahrenheit. A mean value for α is 0.0000065 per degree Fahrenheit, t = change (rise or fall) in temperature in degrees Fahrenheit, and l = length of member in inches.

Redundant Frames.

THE criterion of a redundant frame is that there are members in it which cannot be lengthened or shortened without introducing stresses. The following are true for all simple non-redundant structures :—$b = 2p + 1$. $b = 2j - 3$. $j = p + 2$ where b = number of bars, j = number of joints, and p = number of polygons. The first formula shows that every simply firm structure must have an odd number of bars. In the following formulæ d = number of bars deficient, and e = number of bars redundant or in excess.

Deficient Frames.	Redundant Frames.
$b = 2p + 1 + d.$	$b = 2p + 1 - e.$
$b = 2j - 3 - d.$	$b = 2j - 3 + e.$
$j = p + 2 + d.$	$j = p + 2 - e.$

If a line of symmetry be drawn through any simply firm frame it will not cut more than one bar.

Combined Stresses. Cross Bending and Direct Tension or Compression.

IN the section on "Columns with Eccentric Loads" (pages 326 to 331) any deflection is assumed to be so small as to be inappreciable, which would in general be the case. The treatment given here takes account of the deflection, introducing the term $\pm \frac{Pl^2}{10\,E}$ in the denominator.

In the following investigation let the units be tons and inches.

Let l = Length of member.

A = Area of member.

I = Moment of inertia.

E = Modulus of elasticity.

P = Total direct tensile or compressive load.

e = Eccentricity of P = distance from line of action to neutral axis of member.

f_2 = Unit stress on fibre due to direct load $\frac{P}{A}$.

f_1 = Unit stress on fibre due to bending at section of maximum moment or maximum deflection.

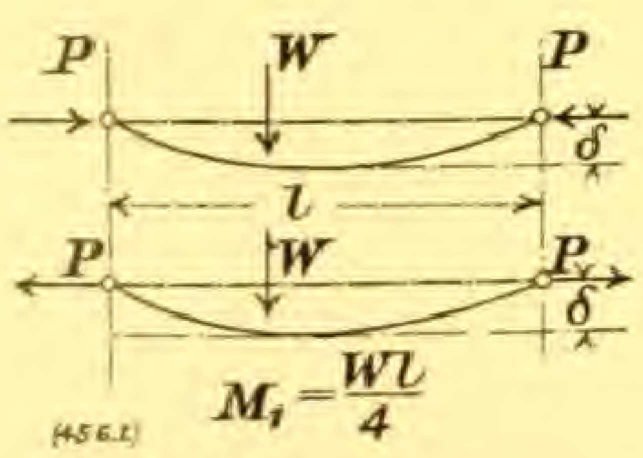
f = Total maximum unit stress on extreme fibre = $f_1 + f_2$.

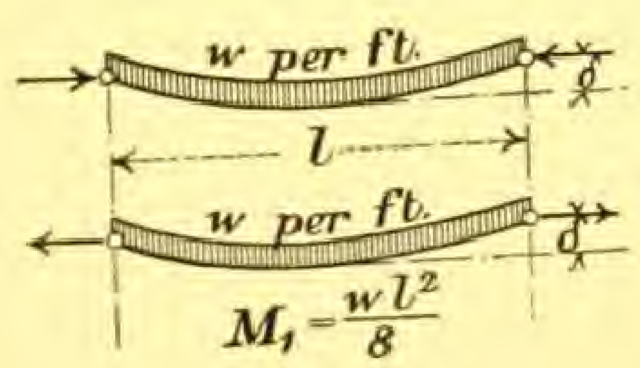
M_1 = Bending moment at point of maximum deflection due to cross bending, external forces, and eccentricity of direct load.

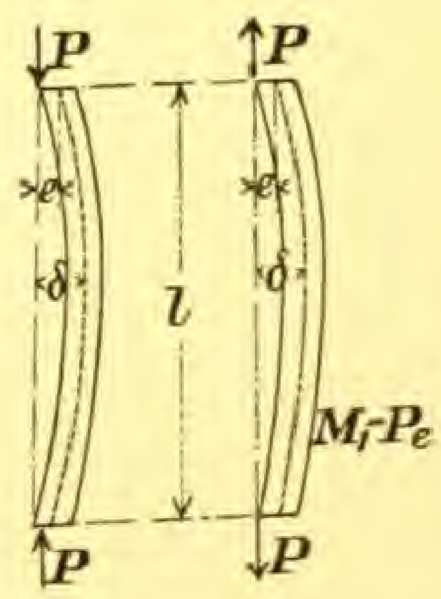
M_2 = Bending moment due to deflection = $P\,\delta$.

y_1 = Distance from neutral axis to extreme fibre in tension or compression, whichever is desired.

δ = Maximum deflection due to all forces acting at once.







We can put $\delta = \frac{f_1 l^2}{K E\, y_1}$ where K is a constant depending upon the end conditions and manner of loading. Substituting δ and reducing $f_1 = \frac{M_1\, y_1}{I \pm \frac{Pl^2}{KE}}$, using the plus sign for compression and minus for tension. This formula is perfectly general, and applies to any section and any loading. It is deduced by Professor Johnson in "Modern Framed Structures," page 173, Eighth Edition. The following values of K are based on the assumption that most cases in practice will correspond closely with uniform loading.

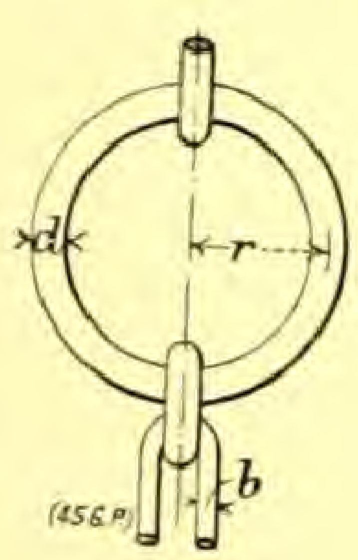
End Conditions of Column, Beam or Bar.	Values of K in $\delta = \frac{f_1 l^2}{K E y_1}$.
Both hinged.	$\frac{48}{5} \doteqdot 10$
One hinged and one fixed.	24
Both fixed.	32

Hinged ends will be more usual, and it is on the side of safety to treat the member thus.

We then have for the total stress

$$f = \frac{P}{A} + \frac{M_1 y_1}{I \pm \frac{P l^2}{10 E}}$$

Strength of Rings.[1]



Let W = Load to be lifted in tons,

b = Diameter of iron in chain,

r = Radius of ring in inches,

d = Diameter of iron in ring,

then $d = \sqrt[3]{r b^2}$,

Maximum bending moment $= \frac{W r}{\pi} = 0.32\ W r$,

Bending moment at C, $M_c = \frac{W r (\pi - 2)}{2 \pi} = 0.182\ W r$

Web Plates.

VARIOUS rules have been suggested for ascertaining the strength of web plates of girders. The generally accepted rules are that the shearing stress per square inch of gross section shall not exceed one-half of the tensile stress per square inch permitted in the flanges of the girder, and that the web plate shall have stiffeners at the bearings and all points of concentrated loading, and at points not further apart than the depth of the girder when the unsupported width of the plate is more than sixty times its thickness. Another common rule is that the stiffeners shall not be placed further apart than 6 ft., nor further apart than the depth of the girder where the stress in tons per square inch of gross section is greater than $\frac{5}{1 + \frac{d^2}{3000\,t^2}}$, in which d is the distance between the edges of the flange angles in inches, and t is the thickness in inches. The stiffeners over the bearings should be designed as columns to carry the whole of the shear at the supports. It is important that the design of the girders should be such that the webs are held up in a truly vertical position to resist the shearing forces.

[1] "ENGINEERING," vol. lvii., page 494.

Buckled Plates.

BUCKLED Plates are usually made from 3 ft. to 6 ft. square, and $\frac{1}{4}$ in. to $\frac{3}{8}$ in. in thickness. They can also now be obtained in long lengths having several buckles to the plate. There is no reliable formula from which the strength of buckled plates can be deduced, but the following table, based on experiments with wrought-iron plates 3 ft. square and 2 in. rise, securely bolted down all round, shows the loads that such plates will safely carry.

Thickness of Plate.	Weight of Plate.	Safe Uniformly Distributed Load per Plate.	Safe Uniformly Distributed Load per Square Foot.
in.	lbs.	tons	tons
$\frac{1}{4}$	90	4.5	0.5
$\frac{5}{16}$	112.5	6.2	0.7
$\frac{3}{8}$	135.0	9.0	1.0

The resistance of a buckled plate, bolted or riveted down all round, is about twice the resistance of the same plate when merely supported all round. If any two opposite edges are not supported the resistance is reduced in the proportion of 8 to 5. Within the limit of safe load the resistance of a buckled plate is practically the same whether the load is resting on the crown of the plate, or is uniformly distributed.

The buckled plates forming the floor of the bridge over the Thames at Westminster are 7 ft. by 3 ft. by $\frac{1}{4}$ in. thick, and have a rise of $3\frac{1}{2}$ in. They were tested by lowering upon the crown of each plate, a block of granite weighing 17 tons, and they sustained this load without injury.

Corrugated Sheets.

THE transverse strength of corrugated sheets is given by the following formula:—

$$w = \frac{99{,}900\, t\, b\, d}{L}$$

where L = Unsupported length of sheet in inches.
t = Thickness of sheet in inches.
b = Breadth of sheet in inches.
d = Depth of corrugations in inches.
w = Breaking weight distributed in pounds.

Strength of Flat Plates.

f = Maximum fibre stress per square inch.
r = Radius in inches.
t = Thickness in inches.
E = Modulus of elasticity, in pounds.
v = Maximum deflection.

w = Load per square inch distributed.
a = Length in inches.
b = Breadth in inches.
f_a = Fibre stress parallel to a.
f_b = Fibre stress parallel to b and $a > b$.

(1) *Circular Plate Supported around its Perimeter.*

$$f = \frac{117\,wr^2}{228\,t^2}$$

$$v = \frac{189\,wr^4}{256\,\mathrm{E}t^3}$$

(2) *Circular Plate Fixed at the Perimeter.*

$$f = \frac{45\,wr^2}{64\,t^2}$$

$$v = \frac{45\,wr^4}{256\,\mathrm{E}t^3}.$$

(3) *Rectangular Plate Fixed at its Edges.*

$$f_a = \frac{b^4wa^2}{2\,(a^4 + b^4)\,t^2}$$

$$f_b = \frac{a^4wb^2}{2\,(a^4 + b^4)\,t^2}$$

$$v = \frac{a^4b^4w}{(a^4 + b^4)\,32\,\mathrm{E}t^3}.$$

(4) *Square Plate Fixed at its Edges.*

$$f = \frac{wa^2}{4\,t^2}$$

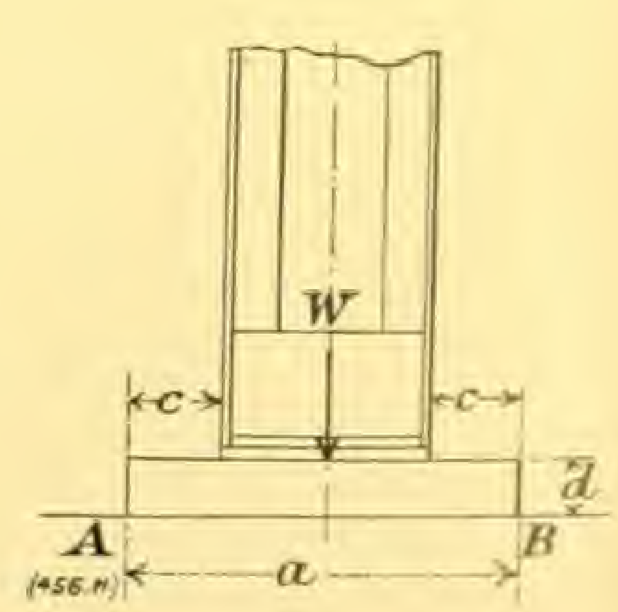
$$v = \frac{wa^4}{64\,\mathrm{E}t^3}.$$

The strength of plates supported on the edges is about $\frac{2}{3}$ the strength of plates fixed. Grashof's "Theorie der Elasticitat und Festigkeit," see Lanza's Mechanics.

Bearing Plates.

THE thickness of bearing plates may be determined as follows:—

Let W = The total load on the plate.
a = Length of plate.
b = Width of plate, at right angles to the paper.
d = Thickness of plate.



Then the load under the projecting portion of the plate $c = \frac{\mathrm{W}}{a}c$ and its moment $\mathrm{M} = \frac{\mathrm{W}c^2}{2\,a}$,

therefore $d = 1.73\,c\sqrt{\frac{\mathrm{W}}{f\,b\,a}}$, where f = stress per square inch on the plate.

Ball Bearings.

PERMISSIBLE pressures for hardened tool steel balls running on surfaces of the same material.

For balls running on flat surfaces, $P = 600\,d^2$.

For balls running in grooves of which the radius is $2/3\,d$, $P = 1200\,d^2$.

P = Permissible load in pounds per ball.

d = Diameter in inches.

"Transactions of the American Society of Civil Engineers," vol. xxxiii.

Roller Bearings.

Let w = the safe load per lineal inch of a roller ;

D = the diameter of roller ; the mean diameter being taken for conical rollers.

Then $w = 300\,D$.

The above formula will give the safe load on rollers from 1 in. to 16 in. diameter, whether they and the plates they are placed between be cast or wrought iron or steel, it being assumed that the working stress is about one-fifth the elastic limit stress for cast iron, and one-third that limit for wrought iron or steel. Should the load to be carried by the rollers be liable to unequal distribution, so that only about one-half of the rollers carry the entire weight, the value of w in the above formula should not exceed

$$w = 200\,D.$$

For rollers supporting swing bridges, the above formulæ may be used for the static load, but only one-half the value of w should be taken for the safe load on the rollers when the bridge is in motion.

These formulæ were obtained from a careful and elaborate series of experiments on soft steel rollers, from 1 in. to 16 in. diameter on soft steel plates, which were undertaken in consequence of the problem being one that cannot be solved analytically for want of more exact knowledge in connection with the distribution of stress and the moduli of elasticity which obtain in cases of this kind.[1]

From these experiments it was found that the elastic limit was reached when the load in pounds was $880\,D$.

The following safe bearing pressures in pounds per lineal inch on rollers are based on experiments by Mr. J. Christie. ("Transactions of the American Society of Civil Engineers," vol. xxxiii.)

	Rollers at Rest.	Rollers in Motion.
Cast iron	$400\,d$.	$200\,d$.
Rolled or cast steel (28 to 32 tons)	$800\,d$.	$400\,d$.
Axle steel (35 to 38 tons)	...	$500\,d$.
Tool steel (60 tons)	...	$800\,d$.
Tool steel, hardened	...	$1000\,d$.

Where d is the diameter in inches. These values are for rollers and bearing surfaces of the same material ; if they are of different materials, the lower values should be used.

[1] "Modern Framed Structures," by Johnson, Bryan and Turneaure. Wiley and Sons, New York. "Transactions of the American Society of Civil Engineers," vol. xxxii., pages 99 and 305.

Bearing Pressure.

Permissible Pressures for Rotating and Sliding Surfaces. (Speed Slow and Intermittent.)[1]

Bearing.	Materials.	Pressure in Pounds per Square Inch.
Pivots for swing bridges	Hardened tool steel on special phosphor-bronze	3500
Trunnion bearings for bascule bridges	Axle steel on phosphor-bronze	2000
Wedges for end lifts	Cast iron on bronze	600
Do. do.	Cast iron on cast iron or rolled steel	500
Screws which transmit motion on projected area of thread	Steel	200

Permissible Pressures for Ordinary Cases. (Moderate Speeds.)[1]

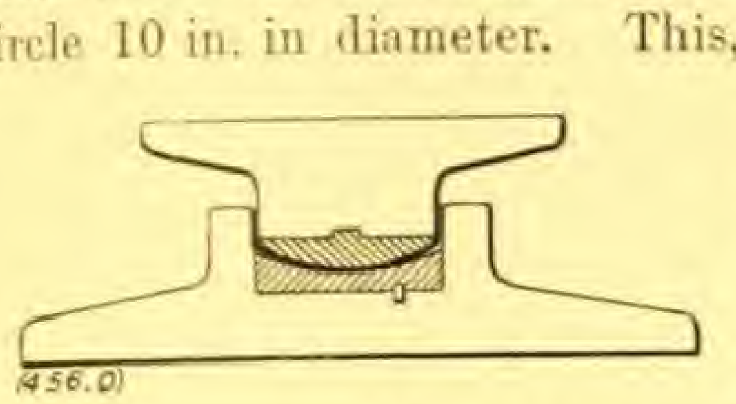
Material.	Pressure in Pounds per Square Inch.
Hardened steel on hardened steel	2000
Hardened steel on bronze	1500
Tool steel (not hardened) on bronze	900
Structural steel on bronze	600
Cast iron on structural steel	400
Cast iron on cast iron	400
Cast iron on crosshead slides, speed not exceeding 600 ft. per minute	50
Cranks, pins, and similar joints with alternating motions	Increase above values 100 per cent.

The above results were obtained with a steel ring of rectangular section held between two cast-iron discs having their bearing surfaces, with gun metal, 12 in. inside and 14 in. outside diameter. The steel ring was held stationary, and the discs rotated at speeds from 50 to 130 revolutions per minute. The bearing surface was oiled through grooves in each face of ring. A load of 75 lb. per square inch was the greatest it would bear at the highest speed without seizing, and 90 lb. at the lowest speed. The coefficient of friction was one-twentieth at 15 lb. per square inch and one-thirtieth at 75 lb.

With a cylindrical bearing the limit of pressure was 100 lb. per square inch with side lubrication, but when it was lubricated on the lower side and allowed to drag the oil the pressure reached 600 lb. per square inch when the surface was taken over the full diameter, if one-sixth of the circle was taken the pressure was 1700 lb. per square inch.

[1] "Transactions of the American Society of Civil Engineers," February, 1907.

In a swing bridge recently constructed with a centre pivot or footstep bearing, 14 in. in diameter, the upper revolving pivot pin had a convex face of tool steel, and the lower fixed cup bearing was also of tool steel with a concave face. The bearing surface was at first equal to the area of a circle 7 in. in diameter. The friction was excessive, and when the bridge was turning the grinding of the pin was very loud. The washers were then taken out and turned to make increased bearing surface, amounting to an area of a circle 10 in. in diameter. This, however, made only a very slight improvement; finally


(456.D)

the bottom washer was changed to phosphor bronze, and the bearing surface increased to that of a circle of 12 in. in diameter. This alteration reduced the friction, and the bridge now turns with great ease and smoothness.

The approximate load upon the centre pivot is 250 tons.

7 in. diameter ; 38.5 square inches for 250 tons = $6\frac{1}{2}$ tons per square inch
10 in. diameter ; 78.5 square inches for 250 tons = 3.2 tons per square inch
12 in. diameter ; 113 square inches for 250 tons = 2.2 tons per square inch

Centre bearing railway turntables usually rest upon three loose discs of sufficient diameter to distribute the pressure. The upper and lower discs are of hardened steel, the middle disc of phosphor bronze, or steel if desired. The discs are placed in a cast-steel oil box. As the discs are 7 in. in diameter, and the weight of the turntable is 15 tons, and locomotive and tender is 100 tons, the pressure on discs is 3 tons per square inch.

Steel, Phosphor-bronze and Gun-metal (Slow Motion).

The safe bearing pressure on flat surfaces of hard steel, phosphor-bronze or gun-metal must not exceed 6000 lb. per square inch for static loads, or 3000 lb. per square inch on well lubricated surfaces in slow motion. For small surfaces, say less than 4 in. in diameter, the above pressures may be increased by one-half. When the pressure approaches 10,000 lb. per square inch, there is danger from abrasion.

Phosphor-bronze, Gun-metal, etc. (Quick Motion).[1]

Safe Working Pressures. Revolving Bearings. Quick Motion.

Type of Bearing.	Maximum Permissible Pressure per Square Inch.
	lb.
Crank pins—Locomotive (alternating pressure) - - - - - -	1500
Crank pins—Marine and stationary (alternating pressure) - - - -	600
Railway car axles - - - - - - - - - - - -	350
Ordinary pedestals—Gun-metal - - - - - - - - -	200
Ordinary pedestals—Good white metal - - - - - - - -	500
Collar and thrust bearing—Gun-metal - - - - - - - -	80
Collar and thrust bearing—Good white metal - - - - - -	200
Collar and thrust bearing—Lignum vitæ - - - - - - -	50
Slide blocks—Cast iron or gun-metal - - - - - - - -	80
Slide blocks—Good white metal - - - - - - - - -	250
Chain and rope pulleys for cranes—Gun-metal bush - - - - -	600 to 1000

[1] "Mechanics Applied to Engineering," page 260. John Goodman. Longmans, 1904.

Friction Experiments.

Friction of Rest. Dry Unlubricated Surfaces (Rennie).

Pressure in Pounds per Square Inch.	Coefficients of Friction.			
	Wrought Iron on Wrought Iron.	Wrought Iron on Cast Iron.	Steel on Cast Iron.	Brass on Cast Iron.
187	0.25	0.28	0.30	0.23
224	0.27	0.29	0.33	0.22
336	0.31	0.33	0.35	0.21
448	0.38	0.37	0.35	0.21
560	0.41	0.37	0.36	0.23
672	Abraded	0.38	0.40	0.23
784	Abraded	Abraded	Abraded	0.23

Rolling Friction.

Pavement.	Speed per Hour in Miles.	Coefficient.	Resistance per Ton in Pounds.	Remarks.
Granite - - -	2.87	0.007	17.41	Experiments with omnibus with various loads. (Committee of Society of Arts, Clark.)
Asphalte - -	3.56	0.0121	27.14	
Wood - - -	3.34	0.0185	41.60	
Macadam (gravelled)	3.45	0.0199	44.48	
Macadam (granite, new) - - -	3.51	0.0451	101.09	

Sliding Friction.

Railway track -	10	0.110	246	Sliding friction of steel tyres on steel rails. (Westinghouse and Galton.)
	15	0.087	195	
	25	0.080	179	
	38	0.051	128	
	45	0.047	114	
	50	0.040	90	

In experiments made with two cast-iron beams the following values of dry sliding and rolling friction were obtained. The beams were 175¼ in. long by 5¾ in. broad, each weighing 1570 lb., and were planed to a smooth surface and accurately levelled. One beam was laid dry upon the other, and afterwards two rollers, 2¼ in. in diameter, were placed 7 ft. apart between them. The force required to start motion was obtained for different loads

Load.	Force to Start Motion.		Coefficient of Friction.	Percentage of Load.	Remarks.
	Total.	Pounds per Ton.			
lb.	lb.	lb.			
1570	450 to 500	650 to 716	0.29 to 0.32	29 to 32	Sliding friction.
2130	475 to 560	500 to 590	0.224 to 0.263	22 to 26	Do.
1570	2.73	3.9	0.00174	0.17	Rolling friction.
2240	5.44	5.44	0.00243	0.24	Do.

A force of 3¾ lb. kept 20 cwt. moving on the rollers for a distance of 45 in., giving a coefficient of friction of 0.00167, or 0.16 per cent. of the load.

A pull of 1 lb. with free rollers is approximately equal to 1 cwt. with dry sliding friction.

In the traversing bridge at Hull the rolling friction was about 10 lb. per ton of moving weight.

In experiments with eleven American turntable swing bridges the friction showed a maximum of 7.94 lb. per 1000 lb. weight of moving structure; one bridge showed as low as 3.53 lb.

From experiments by Crandall and Marston made to determine the resistance to rolling of small rollers, 1 in. to 4 in. in diameter and 1½ in. long, between three horizontal plates of cast iron, wrought iron and steel, the following formulæ were deduced :—

For cast-iron plates

$$\text{Coefficient of friction} = \frac{0.0063}{\sqrt{r}} \text{ for cast-iron rollers,}$$

$$\text{Do.} \quad = \frac{0.0120}{\sqrt{r}} \text{ for wrought-iron rollers,}$$

$$\text{Do.} \quad = \frac{0.0073}{\sqrt{r}} \text{ for steel rollers,}$$

where r = radius in inches.

With wrought-iron plates the friction was 13 per cent. greater, and with steel plates 13 per cent. less.

The rollers were turned and the plates planed, but neither polished.

Economical Depths of Girders.

(1) *Plate Girders.*

Let x = Depth of girder in inches.
W = Weight of girder in pounds.
f = Allowable working stress per square inch on the gross area.
t = Thickness of web.
L = Extreme length of girder.
M = Total bending moment at centre (inch-pounds).

Then

The weight of the web is $\frac{10}{3}Ltx$, and that of the flanges, assuming the flange plates are the required theoretical lengths $= \frac{10}{3}\left\{1.6\frac{ML}{fx}\right\}$;

therefore $$W = \frac{10}{3}Ltx + \frac{10}{3}\left\{1.6\frac{ML}{fx}\right\},$$

and W is a minimum when $x = 1.27\sqrt{\frac{M}{ft}}$.

The above formula is based on the assumption that no portion of the web is included in the flange area. If, however, the resistance of the web is considered,

then $$x = 1.46\sqrt{\frac{M}{ft}}.$$

If the flanges are of constant section throughout as in the case of a rolled joist, then the above formulæ become

$$x = 1.41\sqrt{\frac{M}{ft}} \text{ if resistance of web is neglected,}$$

and $$x = 1.63\sqrt{\frac{M}{ft}} \text{ if resistance of web is considered.}$$

(2) *Lattice Girders.*

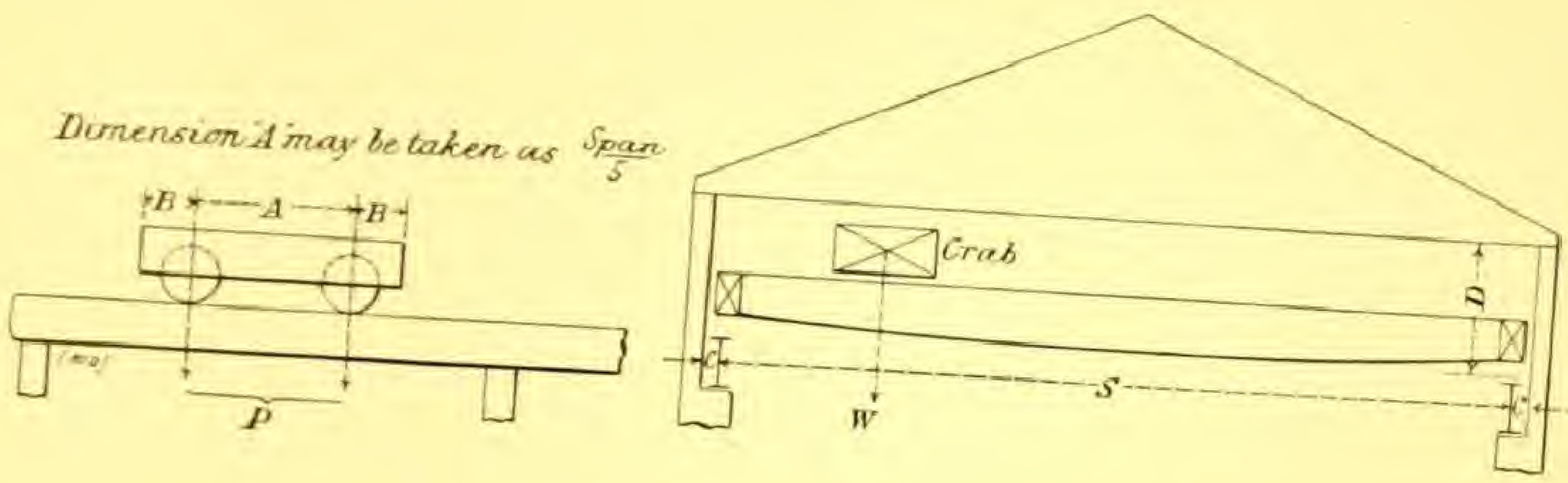
In trusses of the N type the maximum stiffness or minimum deflection is obtained when $$x = \frac{d}{2}\sqrt{\frac{y+2}{y-2}}y$$

where d = length in feet between verticals,
and y = number of bays.

Loads on Girders in Walls.

GIRDERS supporting well built and bonded walls without openings are designed to carry an amount of the wall equal to an equilateral triangle whose base is equal to the span. This does not apply where the walls are green or have openings in them.

Approximate Weights of Travelling Cranes.



Load Lifted = W.	Span of Crane = S.	Maximum Weight on End Wheels = P.	Weight of Crab, Ropes, etc.	Weight of Crane without Crab, etc.	Centre of End Wheels = A.	Overhang of Carriage = B.	End Clearance = C.	Head-Room = D.
	ft.	tons	tons	tons	ft.	ft.	in.	ft.
1 Ton	20	2.5	$\frac{3}{4}$	1.5	4	1	$7\frac{1}{2}$	5
,,	30	3.0	$\frac{3}{4}$	2.5	6	1	$7\frac{1}{2}$	5
,,	40	3.5	$\frac{3}{4}$	3.75	8	1	$7\frac{1}{2}$	5
,,	50	4.0	$\frac{3}{4}$	5.0	10	1	$7\frac{1}{2}$	5
2 Tons	20	3.5	1	1.75	4	1	$7\frac{1}{2}$	5
,,	30	4.0	1	2.75	6	1	$7\frac{1}{2}$	5
,,	40	4.75	1	4.0	8	1	$7\frac{1}{2}$	5
,,	50	5.5	1	5.25	10	1	$7\frac{1}{2}$	5
3 Tons	20	5.0	$1\frac{1}{2}$	2.0	4	1	8	5
,,	30	6.0	$1\frac{1}{2}$	3.5	6	1	8	5
,,	40	7.0	$1\frac{1}{2}$	5.0	8	1	8	5
,,	50	8.0	$1\frac{1}{2}$	7.0	10	1	8	5

Load Lifted = W.	Span of Crane = S.	Maximum Weight on End Wheels = P.	Weight of Crab, Ropes, etc.	Weight of Crane without Crab, etc.	Centre of End Wheels = A.	Overhang of Carriage = B.	End Clearance = C.	Head-Room = D.
	ft.	tons	tons	tons	ft.	ft.	in.	ft.
5 Tons	20	7.5	2	2.5	4	1	8	$5\frac{1}{2}$
,,	30	8.5	2	4.0	6	1	8	$5\frac{1}{2}$
,,	40	9.5	2	6.0	8	1	8	$5\frac{1}{2}$
,,	50	10.5	2	8.0	10	1	8	$5\frac{1}{2}$
$7\frac{1}{2}$ Tons	20	10.0	3	3.0	4	$1\frac{1}{4}$	8	$5\frac{1}{2}$
,,	30	11.5	3	5.0	6	$1\frac{1}{4}$	8	$5\frac{1}{2}$
,,	40	13.0	3	7.0	8	$1\frac{1}{4}$	8	$5\frac{1}{2}$
,,	50	14.5	3	9.0	10	$1\frac{1}{4}$	8	$5\frac{1}{2}$
10 Tons	20	13.5	4	4.0	6	$1\frac{1}{4}$	8	$5\frac{1}{2}$
,,	30	15.5	4	6.0	6	$1\frac{1}{4}$	8	$5\frac{1}{2}$
,,	40	17.5	4	8.0	8	$1\frac{1}{4}$	8	$5\frac{1}{2}$
,,	50	18.5	4	10.0	10	$1\frac{1}{4}$	8	$5\frac{1}{2}$
15 Tons	20	19.0	5	5.0	6	$1\frac{1}{4}$	8	$5\frac{3}{4}$
,,	30	21.0	5	6.5	6	$1\frac{1}{4}$	8	$5\frac{3}{4}$
,,	40	23.0	5	9.0	8	$1\frac{1}{4}$	8	$5\frac{3}{4}$
,,	50	25.0	5	11.0	10	$1\frac{1}{4}$	8	$5\frac{3}{4}$
20 Tons	30	28.0	7	7.0	6	$1\frac{1}{4}$	8	6
,,	40	30.0	7	10.0	8	$1\frac{1}{4}$	8	6
,,	50	32.0	7	13.0	10	$1\frac{1}{4}$	8	6
,,	60	34.0	7	16.0	12	$1\frac{1}{4}$	8	6
25 Tons	30	35.0	8	9.0	8	$1\frac{1}{2}$	9	$6\frac{1}{2}$
,,	40	37.0	8	12.0	8	$1\frac{1}{2}$	9	$6\frac{1}{2}$
,,	50	39.0	8	15.0	10	$1\frac{1}{2}$	9	$6\frac{1}{2}$
,,	60	41.0	8	18.0	12	$1\frac{1}{2}$	9	$6\frac{1}{2}$
30 Tons	30	41.0	10	10.5	8	$1\frac{1}{2}$	9	$7\frac{1}{2}$
,,	40	44.0	10	14.0	8	$1\frac{1}{2}$	9	$7\frac{1}{2}$
,,	50	47.0	10	17.5	10	$1\frac{1}{2}$	9	$7\frac{1}{2}$
,,	60	50.0	10	21.0	12	$1\frac{1}{2}$	9	$7\frac{1}{2}$
35 Tons	40	50.0	12	15.5	8	$1\frac{1}{2}$	10	$7\frac{1}{2}$
,,	50	53.0	12	19.0	10	$1\frac{1}{2}$	10	$7\frac{1}{2}$
,,	60	56.0	12	22.5	12	$1\frac{1}{2}$	10	$7\frac{1}{2}$
,,	70	59.0	12	26.0	14	$1\frac{1}{2}$	10	$7\frac{1}{2}$

Load Lifted = W.	Span of Crane = S.	Maximum Weight on End Wheels = P.	Weight of Crab, Ropes, etc	Weight of Crane without Crab, etc.	Centre of End Wheels = A.	Overhang of Carriage = B.	End Clearance = C.	Head-Room = D.
	ft.	tons	tons	tons	ft.	ft.	in.	ft.
40 Tons	40	57.0	14	17.0	8	$1\frac{1}{2}$	10	$7\frac{1}{2}$
,,	50	60.0	14	20.5	10	$1\frac{1}{2}$	10	$7\frac{1}{2}$
,,	60	63.0	14	24.0	12	$1\frac{1}{2}$	10	$7\frac{1}{2}$
,,	70	66.0	14	27.5	14	$1\frac{1}{2}$	10	$7\frac{1}{2}$
,,	80	69.0	14	31.0	16	$1\frac{1}{2}$	10	$7\frac{1}{2}$
45 Tons	40	64.0	15	19.0	8	$1\frac{1}{2}$	12	8
,,	50	67.0	15	23.0	10	$1\frac{1}{2}$	12	8
,,	60	70.0	15	27.0	12	$1\frac{1}{2}$	12	8
,,	70	73.0	15	31.0	14	$1\frac{1}{2}$	12	8
,,	80	76.0	15	35.0	16	$1\frac{1}{2}$	12	8
50 Tons	40	70.0	17	21.0	8	2	12	8
,,	50	74.0	17	25.5	10	2	12	8
,,	60	78.0	17	30.0	12	2	12	8
,,	70	82.0	17	34.5	14	2	12	8
,,	80	86.0	17	39.0	16	2	12	8
60 Tons	40	84.0	19	25.0	8	2	12	$8\frac{1}{2}$
,,	50	88.0	19	30.0	10	2	12	$8\frac{1}{2}$
,,	60	92.0	19	36.0	12	2	12	$8\frac{1}{2}$
,,	70	96.0	19	42.0	14	2	12	$8\frac{1}{2}$
,,	80	100.0	19	48.0	16	2	12	$8\frac{1}{2}$
70 Tons	40	98.0	22	30.0	8	2	12	$9\frac{1}{2}$
,,	50	102.0	22	36.0	10	2	12	$9\frac{1}{2}$
,,	60	106.0	22	42.0	12	2	12	$9\frac{1}{2}$
,,	70	110.0	22	48.0	14	2	12	$9\frac{1}{2}$
,,	80	114.0	22	55.0	16	2	12	$9\frac{1}{2}$
80 Tons	50	116.0	26	40.0	10	3	14	$9\frac{1}{2}$
,,	60	121.0	26	46.0	12	3	14	$9\frac{1}{2}$
,,	70	126.0	26	54.0	14	3	14	$9\frac{1}{2}$
,,	80	131.0	26	62.0	16	3	14	$9\frac{1}{2}$
,,	90	136.0	26	70.0	18	3	14	$9\frac{1}{2}$

Load Lifted = W.	Span of Crane = S.	Maximum Weight on End Wheels = P.	Weight of Crab, Ropes, etc.	Weight of Crane without Crab, etc.	Centre of End Wheels = A.	Overhang of Carriage = B.	End Clearance = C	Head-Room = D.
	ft.	tons	tons	tons	ft.	ft.	in.	ft.
90 Tons	50	131.0	30	46.0	10	3	14	10
"	60	137.0	30	54.0	12	3	14	10
"	70	143.0	30	63.0	14	3	14	10
"	80	149.0	30	72.0	16	3	14	10
"	90	155.0	30	81.0	18	3	14	10
100 Tons	50	148.0	34	52.0	10	3	14	10
"	60	155.0	34	62.0	12	3	14	10
"	70	162.0	34	72.0	14	3	14	10
"	80	169.0	34	82.0	16	3	14	10
"	90	176.0	34	92.0	18	3	14	10
"	100	183.0	34	102.0	20	3	14	10

REMARKS.—Cranes not given in the above Tables may be proportioned from the data given.

Distribution of Crane Wheel Loads on Top Flanges of Gantry Girders.

THE concentrated loads from the wheels of travelling cranes are distributed along a length of the top flange by the combined stiffness of the rail and flange. The distribution depends upon the sections adopted for the rail and flange, and may be assumed to have a length of from four to six times the total depth of flange angles and rail. This vertical load must be combined with the horizontal shear in determining the rivets connecting the web plate to the flange angles.

Lateral Forces on Top Flanges of Gantry Girders.

THE top flanges should have sufficient width and be of such a form as will resist the lateral forces from cross travel of the crab or from dragging weights across the shop floor. To provide for these forces the flanges of each girder should be designed to resist a horizontal force of at least one-twentieth of the lifting capacity of the crane in addition to the direct stresses from the vertical loading.

Water Pressure.

THE pressure of water in motion on staging and the piers of a bridge, or other structure, may be approximately ascertained by the following formulæ.

Surface and Bottom Velocity of Rivers.

Let v = Velocity of water at surface in inches per second,

then Velocity at bottom $= (v + 1) - 2\sqrt{v}$ and

Mean velocity $= (v + 0.5) - \sqrt{v}$.

Obstructions in Rivers.

Let V = Velocity of river previous to obstruction in feet per second,

A = Sectional area of river unobstructed in square feet,

a = Sectional area of river at obstruction in square feet,

R = Rise of water caused by the obstruction in feet,

then $$R = \left\{\frac{V^2}{58.6} + 0.05\right\}\left\{\left(\frac{A}{a}\right)^2 - 1\right\}.$$

Force of Water in Motion.

Let V = Velocity of current (feet per second),

P = Pressure on a plane normal to the current in pounds per square foot,

then $P = 1.8\ V^2$ for fresh water,

$= 1.85\ V^2$ for salt water.

The pressure P may be taken as acting at one-third the depth of the river from the surface.

For the pressure of fluids on inclined surfaces and the relative resistance of surfaces of different form, see page 389.

Wind Pressure.

LET P = wind pressure in pounds per square foot, on a flat surface, normal to the direction of the wind, V = velocity in miles per hour;

Then, according to careful experiments made in the open air at the National Physical Laboratory (P.I.C.E., vol. clxxi.), $P = 0.0032\ V^2$.

This formula gives the pressures for different velocities shown in the following Table:—

Velocity in Miles per Hour.	Pressure per Square Foot in Pounds.	Remarks.
10	0.32	Gentle.
20	1.28	Light breeze.
30	2.88	Moderate wind.
40	5.12	High wind.
50	8.00	Gale.
60	11.52	Storm.
70	15.68	Heavy storm.
80	20.48	Violent storm.
90	25.92	Hurricane.
100	32.00	Violent hurricane.
120	46.08	—
140	62.72	—

If the velocity of the wind should be variable, the resulting pressure may be considerably augmented.

The experiments referred to above also showed that the negative pressure on the leeward side of a triangular roof may be greater than the positive pressure on the windward side of the roof slope, and the negative pressure on the leeward side of the building may be as great as one-third of the positive pressure on the windward side.

The experiments of Stevenson form a guide to varying pressures at different levels. They show the varying velocities of wind at different levels in an open space.

Feet above ground.	5	9	15	25	52
Velocities in miles per hour.	4	6	6	6.5	7.5
	7	17	18	21	23
	13	23	25	30	32
	19	28	31	35	40
	26	32	34	37	43
Average	13.8	21.2	22.8	25.9	29.1

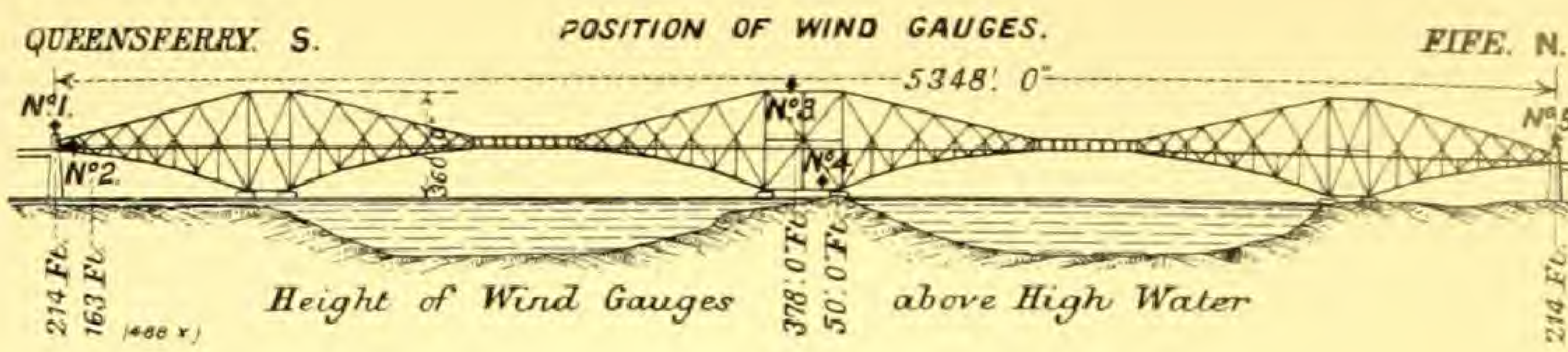
During the construction of the Forth Bridge, careful records were made of the wind pressures which occurred during the years 1883 to 1890. The following particulars of these experiments are taken from "ENGINEERING," of February 28, 1890. The large fixed gauge was a board 20 ft. long by 15 ft. high, or 300 square feet area. This was placed on the top of the old castle, on the island of Inchgarvie, in the middle of the Firth of Forth. It was set vertically, facing east and west. In the centre and top corner of this board small gauges were fixed, with separate recording apparatus. Each had an area of 1½ square feet, and recorded local pressures from gusts. By the side of this board there was another gauge, at a distance of about 8 ft. It consisted of a circular plate, 1½ square feet area, facing east and west. A gauge of the same dimensions, but with the disc attached to the short arm of a double vane, so that it would always face the wind, was also set up.

In the following Table the most violent gales which occurred during the construction of the Forth Bridge are given, with the pressures recorded on the gauges.

RECORDS OF WIND PRESSURES ON INCHGARVIE DURING VIOLENT GALES.

Date.	Pressure in Pounds per Square Foot.					Direction of Wind.
	Revolving Gauge, 1.5 sq. ft.	Small Fixed Gauge, 1.5 sq. ft.	Large Fixed Gauge, 300 sq. ft.	In Centre of Large Gauge. 1.5 sq. ft.	Right-hand Top of Large Gauge.	
October 27, 1884..	29	23	18	...	...	S.W.
October 28, 1884..	26	29	19	...	...	S.W.
March 20, 1885 ...	30	25	17	...	...	W.
December 4, 1885	25	27	19	...	...	W.
March 31, 1886 ...	26	31	19	28.5	22.0	S.W.
February 4, 1887...	26	41	15	...	...	S.W.
January 5, 1888 ...	27	16	7	...	...	S.E.
November 17, 1888	35	41	27	...	...	W.
November 2, 1889	27	34	12	...	...	S.W.
January 19, 1890...	27	28	16	...	...	S.W.
January 21, 1890...	26	38	15	...	...	W.
January 25, 1890..	27	24	18	23½	22	S.W. by W.
Average	27.6	29.8	16.9	—	—	—

Since the completion of this bridge, in 1890, the maximum wind pressures have been recorded. The wind gauges are plate-pressure revolving gauges with double vanes, having an area of 1½ square feet each. They are placed as shown in the sketch. The distance between the extreme gauges is about 1 mile, and the difference of elevation about 350 ft. The highest pressure recorded is 65 lb. per square foot, on gauge No. 3. The following records are from the "Transactions of the Junior Institution of Engineers," 1906:—



RECORDS OF STORMS AT THE FORTH BRIDGE.

PRESSURES IN POUNDS PER SQUARE FOOT.

Date.	Position of Wind Gauges.					Remarks.
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	See Sketch above for Position of Gauges.
January 26, 1901...	25	15	65	...	...	
November 23, 1901	55	50	60	...	55	
December 13, 1902	31	27.5	18	...	34	
January 10, 1903...	25	20	60	15	27.5	
January 31, 1903...	29	19.5	65	...	26	
March 18, 1903 ...	25	20	31	20	29	W.S.W.
March 21, 1903 ...	20	20	54	10	22.5	W. and S.W.
March 26, 1904 ...	32	20	52	...	27	
December 29, 1904	22.5	22.5	...	...	32.5	
January 21, 1905...	30	21	...	...	23	
March 18, 1905 ...	32.5	32.5	60	...	42	For gale of March 15
February 28, 1905	20	22	38	10	20	
January 11, 1906...	23.5	20	30	10	25	
January 26, 1906...	...	...	59	15	...	S.W.
February 8, 1906...	25	15	55	10	25	
Average	28.0	23.0	50.0	13.0	30.0	—

In America the building laws of New York, Boston, and Chicago require that steel buildings shall be designed for a horizontal wind pressure of 30 lb. per square foot; and in recent German specifications for the design of tall chimneys, the wind pressures to be allowed for are given as follows:

Rectangular chimneys, 26 lb. per square foot.
Circular chimneys, 17.4 lb. per square foot.
Octagonal chimneys, 18.4 lb. per square foot.

It would appear, therefore, that 30 lb. per square foot is sufficient pressure to provide for in ordinary cases, except for buildings in very exposed positions. For buildings more or less sheltered by surrounding objects, 20 lb. per square foot is sufficient for the structure as a whole, where such do not exceed 30 ft. in height; but 30 lb. per square foot should be allowed in designing separate portions of the building presenting small areas. In the design of buildings it is sufficient to assume an average steady wind pressure of 30 lb. per square foot in estimating the wind load on any member of the framework supporting an area of 300 square feet and under, and reduce this pressure by 1 lb. per square foot for every 100 square feet in excess of this amount to a minimum of 20 lb. per square foot. The building should, in addition, resist overturning with a steady pressure of 50 lb. per square foot. The uplifting effect of the wind need not be considered, except in the case of open sheds in an exposed position.

It has been shown by experiment that the wind pressure upon surfaces more or less sheltered by those immediately in front of them, varies very considerably according to the distance apart of the surfaces, but in no case does the area affected by the wind exceed 1.8 times the area of the surface directly fronting the wind.

Relative Wind Resistance of Various Surfaces.

The values given by various experimenters are as follows[1]:

Flat plate	1
Parachute (concave surface), depth = $\frac{\text{diameter}}{3}$	1.2 to 2
Sphere	0.36 to 0.41
Elongated projectile	0.5
Cylinder	0.54 to 0.57
Wedge (base to wind)	0.8 to 0.97
Wedge (edge to wind), vertex angle 90 deg.	0.6 to 0.7
Cone (base to wind)	0.95
Cone (apex to wind), vertex angle 90 deg.	0.69 to 0.72
Cone (apex to wind), vertex angle 60 deg.	0.54
Lattice girders	about 0.8

Pressure on Inclined Surfaces: Duchemin's Formulæ.

Let P = Intensity of pressure on a plane normal to the direction of the wind.

P_n = Intensity of normal pressure on a plane inclined at angle θ to the horizontal

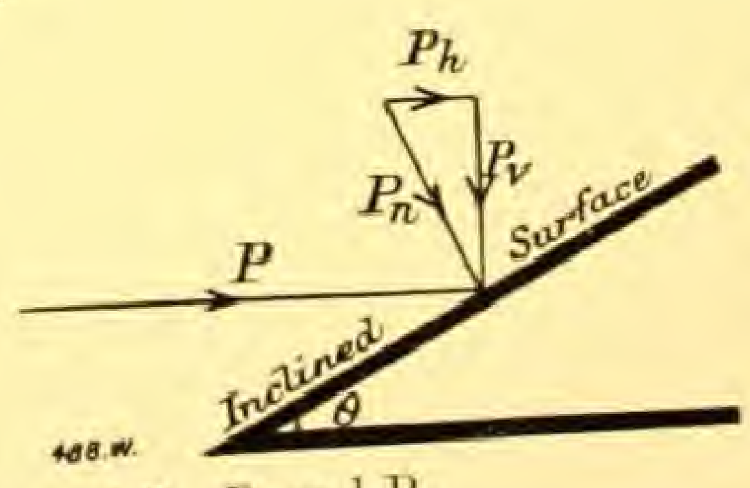
P_v = Vertical component of normal pressure.

P_h = Horizontal component of normal pressure.

Then $$P_n = P\,\frac{2\sin\theta}{1+\sin^2\theta}$$

$$P_v = P\,\frac{2\sin\theta\cos\theta}{1+\sin^2\theta}$$

$$P_h = P\,\frac{2\sin^2\theta}{1+\sin^2\theta}$$



For P = unity, the following are the values of P_n, P_v and P_h.

[1] Goodman's "Mechanics Applied to Engineering," page 503, 1906.

Table for Duchemin's Constants for P_n, P_v, P_h.

Angle θ.	Deg. 5	Deg. 10	Deg. 15	Deg. Min. 18 26	Deg. 20	Deg. Min. 21 48	Deg. 25	Deg. Min. 26 34	Deg. 30
P_n	.173	.337	.485	.575	.612	.653	.717	.745	.800
P_v	.172	.332	.468	.546	.575	.607	.650	.665	.692
P_h	.015	.058	.125	.182	.209	.242	.303	.333	.400

Angle θ.	Deg. Min. 33 41	Deg. 35	Deg. 40	Deg. 45	Deg. 90
P_n	.848	.863	.910	.943	1
P_v	.707	.709	.696	.667	0
P_h	.470	.496	.584	.667	1

Example: If P = 30 lb. per square foot, and θ = 30 deg.

$P_n = 30 \times 0.8 = 24$. $P_v = 30 \times 0.692 = 20.7$

$P_h = 30 \times 0.4 = 12.$

$\frac{\text{Rise}}{\text{Span}}$	Angle θ. deg. min.
$\frac{1}{6}$	18 26
$\frac{1}{5}$	21 48
$\frac{1}{4}$	26 34
$\frac{1}{3}$	33 41
$\frac{1}{2}$	45 —

Roof Drainage.[1]

Vertical length of down pipe, about 31 ft. 6 in., with about five bends of 45 deg. each.

Area of roof drained by one down pipe—4945 square feet.

Gutter level throughout, and hydraulic gradient during flow was about 1 in 600 in a length of 106 ft.

Internal Diameter of Down Pipes, Inches.	Fall of Level of Water in Gutter.	Contents Discharged in Cubic Inches.	Observed Mean Duration of Flow in Seconds.	Discharge in Cubic Inches per Second.
5	6 in. to 5 in.	18,975	11.5	1650.0
5	5 in. to 4 in.	18,975	26.5	716.0
5	4 in. to 3 in.	18,975	37.0	512.8
5	3 in. to 2 in.	18,975	66.5	285.3
5	6 in. to 2 in.	75,900	141.5	536.4

[1] "Notes on Construction in Mild Steel," by H. Fidler.

Temperature.

THE records of temperature in Great Britain which have been made at Greenwich show the average range for twenty years to be only 11 deg. Fahr. below and 13 deg. Fahr. above the mean temperature. The expansion of steel due to this range would only be about $\frac{3}{16}$ in. per 100 ft. The maximum annual range of shade temperature may be, however, 85 deg. Fahr., and in the sun even as great as 137 deg. Fahr. These temperatures would theoretically cause steel to expand $\frac{5}{8}$ in. and 1 in. respectively in a length of 100 ft. The observed movement due to temperature of the lattice iron viaduct carrying the Caledonian Railway over the Clyde at Glasgow is only 2 in. for a length of 376 ft., or a little over $\frac{1}{2}$ in. in 100 ft. This corresponds to a range of temperature of 70 deg. Fahr. In the Forth Bridge the observed range is $\frac{7}{16}$ in. per 100 ft. It is not usual to provide for expansion and contraction in the steelwork used in buildings, and there are many important structures, such as station roofs, where it has been entirely neglected. In most cases there would be no risk in carrying masonry or brick walls on steel girders of very considerable length, but no general rule can be given, and each case must be decided in accordance with its own conditions. Many stone buildings and arches have high stresses upon them due to the changes of temperature. In iron or steel bridges, although longitudinal movement is generally provided for, no allowance is usually made for expansion and contraction in the direction of the width of the structure, however great this may be.

Effect of Temperature on Metallic Structures.

Difference of temperature between top and bottom booms of a girder placed so that the top boom is subject to the direct rays of the sun, and the bottom boom in the shade, is less than the difference of temperature registered in the sun and shade, as the booms are connected together either by a web plate or diagonal bars, which act as conductors of heat, distributing the heat equally throughout the structure. The temperature of the structure is always higher than that of the surrounding atmosphere. The valuable property possessed by ductile metals of slowly yielding or flowing under great pressures, without apparent diminution of strength, must be taken into account when considering the stresses due to variations of temperature. The forces due to unequal temperatures on different parts of a metallic frame from atmospheric causes act very slowly, and consequently are those best adapted to bring into action the flowing properties of the metals, and cause the different parts of the frame to adjust themselves to the gradual stresses brought upon them.

Records of Structures.

For a range of temperature of 50 deg. Fahr. the rise in the crown of one of the cast-iron arches of Southwark Bridge was about $1\frac{1}{4}$ in., the length of the chord of the intrados being 246 ft., and the versed sine 23 ft. 1 in. The length of the segmental arch is 3021 in. At the Britannia Tubular Bridge[1] the temperature of the upper part of the bridge was

[1] "Tubular Bridges," by Edwin Clark.

found during hot sunshine to be 120 deg. Fahr., while in winter the snow lying on the bridge was found to have a temperature of 16 deg. Fahr. The total range of temperature was 104 deg. Fahr. An increase of temperature of 26 deg. Fahr. (from 32 deg. to 58 deg.) gives an increase in length of $3\frac{1}{4}$ in. in the whole bridge. The expansion is thus $\frac{1}{8}$ in. for each degree, or $\frac{1}{1456}$th part of the whole length. It attains its maximum and minimum usually at 3 p.m. and 3 a.m. It sometimes bends $2\frac{1}{2}$ in. laterally and $2\frac{1}{2}$ in. vertically when the sun shines on one side or on top of the tube. On very hot sunny days the lateral motion has been so much as 3 in., and the rise and fall 2 in. A heavy train deflects the tubes only $\frac{3}{10}$ in., and a violent gale $\frac{1}{4}$ in. The heaviest gales do not produce as much motion as ten men. The effects of the sun and wind have reference to the tubes before they were connected in the towers; after they had been connected the heaviest gales do not vibrate them more than $\frac{1}{4}$ in., and the sun does not move them more than $\frac{1}{4}$ in. or $\frac{3}{8}$ in.

The Mechanical Force of Heat.[1]

Materials.	Expansion or Contraction of Bar 1 Ft. Long for a Variation of 1 Deg. Fahr.	Extension or Compression Caused by a Load of 1 Ton on a Bar 1 Ft. Long and 1 Square Inch in Section.	Force Exerted in Expanding or Contracting by a Bar 1 Square Inch in Section for a Variation of 1 Deg. Fahr.	Variation in Temperature Required to Produce a Force of 1 Ton in a Bar 1 Square Inch in Section.
	ft.	ft.	lbs.	deg., Fahr.
	a	b	c	$b \div a = d$
Brass, Cast	0.000010434	0.0002463	94.8864	23.6
Brass, Wire	0.000010720	0.0001600	150.0800	14.9
Copper	0.000009540	0.0001433	149.1168	15.0
Iron, Cast (in tension)	0.000006220	0.0001575	88.4576	25.3
Iron, Cast (in compression)	0.000006220	0.0001600	87.0688	25.7
Iron Wrought	0.000006780	0.0000800	189.8400	11.8
Iron Wrought, Soft	0.000006780	0.0001000	151.8720	14.8
Iron Wrought, Wire	0.000006860	0.00007225	212.7328	10.5
Steel, untempered	0.000005995	0.0000540	248.6848	9.9
Steel, annealed	0.000006886	0.0000779	197.9936	13.3
Glass	0.000050000	0.0002240	499.9680	4.5
Lead	0.000016200	0.0031584	11.4912	195.0
Tin, Cast	0.000012500	0.000393568	71.1424	31.5

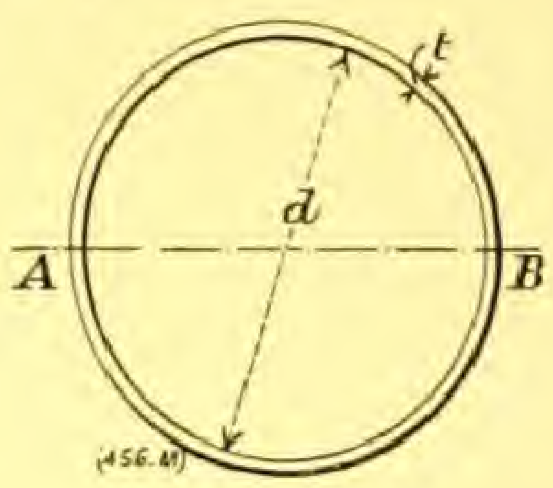
It will be observed that for the same variation of temperature, glass exerts a greater force in expanding and contracting than any of the metals.

[1] "Expansion of Structures by Heat," by John Kelly.

Hollow Cylinders.

Thin Hollow Cylinder.—Stresses from internal or external fluid pressure.

Let d = Inside diameter of cylinder in inches.
t = Thickness of metal of cylinder in inches.
p = Fluid pressure in pounds per square inch.
f = The mean hoop-tension at A or B per square inch.



Then $f = \dfrac{p\,d}{2\,t}$.

If the pressure p is external, then $f = \dfrac{p\,d_1}{2\,t}$ where d_1 = external diameter.

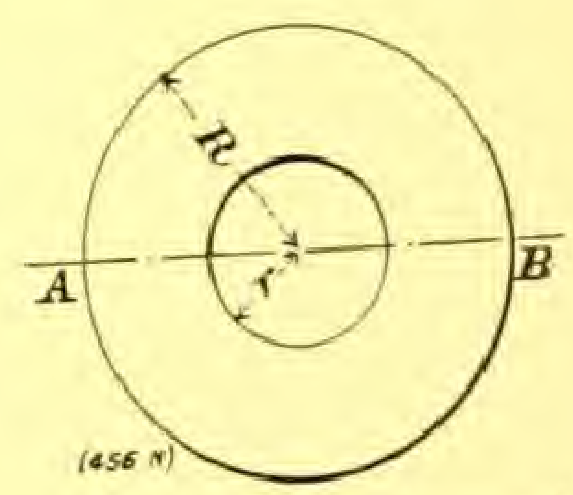
For thin cylinders with closed ends the actual stress within the elastic limit is only $\frac{7}{8}$ that given by the above formula, owing to the longitudinal tension partly neutralising the hoop-tension.[1]

Thin Hollow Sphere.—For a thin hollow sphere the above formulæ become

$$f = \frac{p\,d}{4\,t} \text{ for internal pressure, and } f = \frac{p\,d_1}{4\,t} \text{ for external pressure.}$$

Thick Hollow Cylinder.[2]

Let R = External radius of the cylinder.
r = Internal " "
p = The fluid pressure.
f = The maximum unit stress at A or B.



Then $f = p\left(\dfrac{R^2 + r^2}{R^2 - r^2}\right)$ and $\dfrac{R}{r} = \sqrt{\dfrac{f + p}{f - p}}$.

One important consequence of this equation is that if the internal pressure p is equal to or greater than f, no thickness, however great, will enable the cylinder to resist the pressure. When this occurs the cylinders are constructed of concentric rings, built together, the outer ones being shrunk on the inner ones.

Thick Hollow Sphere.—For a thick hollow sphere the above formulæ become

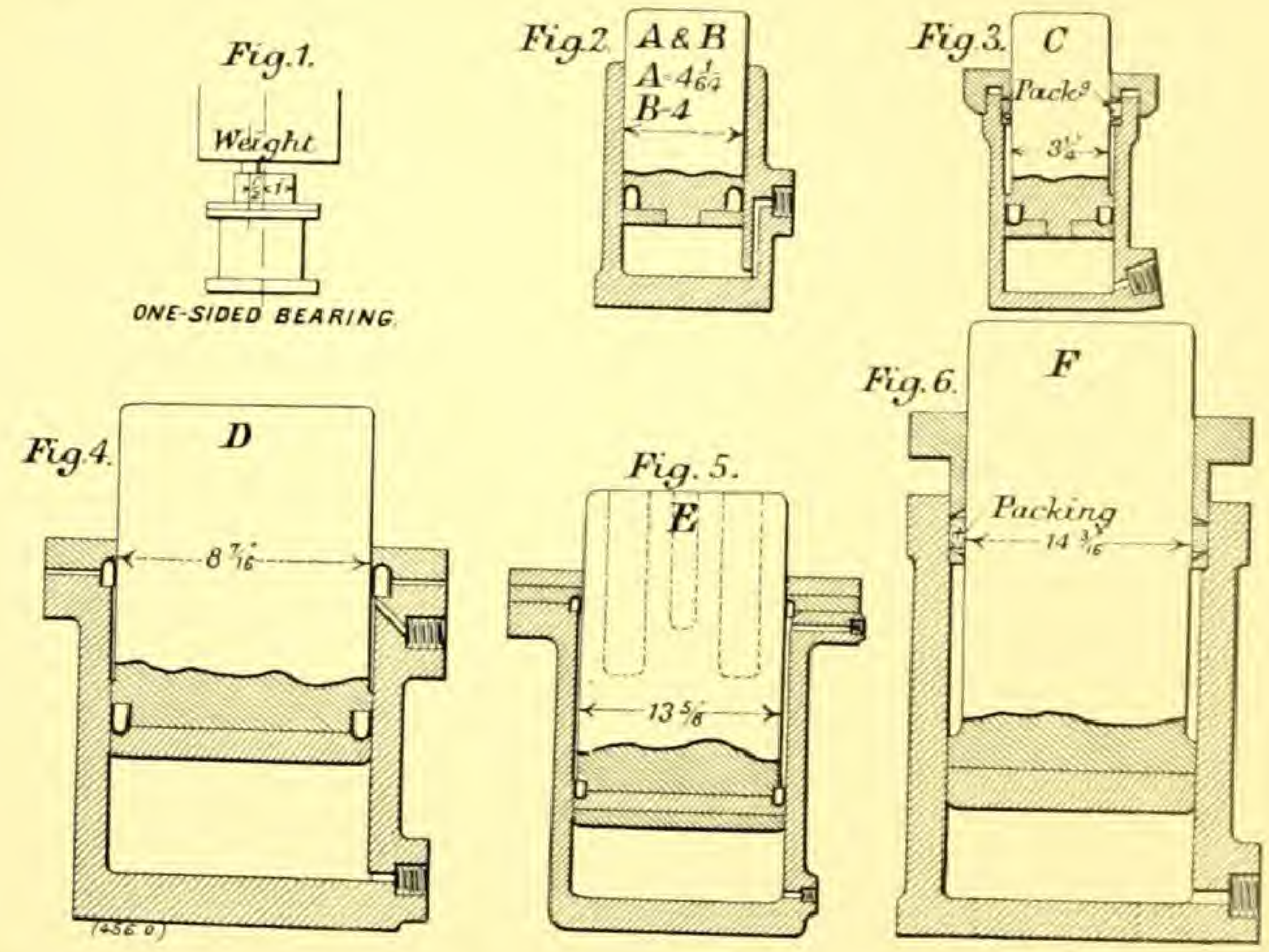
$$f = p\left(\frac{R^3 + 2\,r^3}{2\,R^3 - 2\,r^3}\right) \text{ and } \frac{R}{r} = \sqrt[3]{\left(\frac{2f + 2p}{2f - p}\right)}.$$

The Friction of Hydraulic Rams.

THE experiments recorded in the following Tables were made by the late Mr. J. E. Tuit, M.I.C.E., to determine the probable loss of power due to the friction of the cup leathers, or packings of hydraulic rams. In order that the conditions of practice might be represented, the jacks chosen to be tested had been in everyday

[1] "Mechanics Applied to Engineering," John Goodman, page 270 (1899).
[2] Rankine, "Civil Engineering," page 229 (21st edition).

use for some time at the Forth Bridge. The loss of power due to friction, in this case, is given in Column 6 of the annexed Table. Again, since in practice the weight to be raised, or the external resistance offered to its action, is seldom uniformly distributed over the upper surface of the ram, a strip of iron 1 in. wide and $\frac{1}{2}$ in. thick, was introduced between the upper surface of the ram and the plate it was pressing against, as shown in Fig. 1, thus representing a local load not coinciding with the axis of the ram. The loss of power due to friction, in this case, with an unequally distributed load, is given in Column 8 of the Table. In most cases the rams were again tested after having their cup leathers renewed, and being thoroughly cleaned and greased. The results for loss of power due to friction are given in Column 10.



The jacks A, B and C, Figs. 2 and 3, were tested by placing them singly in a Wicksteed testing machine, arranged for experimenting on specimens in compression, and the pressure of water recorded that was just sufficient to cause each to raise the loaded arm of the machine. In order to obtain a steadier pressure than can be obtained by hand pumps, a small jack was inverted over one 9 in. in diameter. The latter was supplied from a large accumulator with water at a pressure of a 1000 lb. per square inch, and was capable of forcing the water from the jack above it to the one being tested, at a pressure of about 3 tons per square inch. This pressure multiplier stood near the testing machine.

In testing large jacks it was not possible to place them directly in the testing machine. The method adopted was to observe, on the application of certain loads, what pressure was just sufficient to force in the rams of 4 in. diameter jacks. These jacks were then placed over one of the larger ones, in which the pressure was recorded when it was also capable of forcing the small rams in under similar conditions.

The gauges used for registering the pressures were, in all cases, tested before and after the experiments.

HYDRAULIC TESTS.

1 Mark on Rams (see Figs. 2 to 6).	2 Diameter of Ram.	3 Area of Ram.	4 Weight Raised.	5 Pressure Required per Square Inch. Ram in Ordinary Working Condition (Good Bearing).	6 Loss per Cent.	7 Pressure Required per Square Inch. Ram in Ordinary Working Condition (One-sided Bearing).	8 Loss per Cent.	9 Pressure Required per Square Inch after New Cup Leather, and being Regreased (Good Bearing).	10 Loss per Cent.	Remarks.
	in.	sq. in.	tons	cwt.		cwt.		cwt.		
A	$4\frac{1}{64}$	12.66	6	...	...	$10\frac{1}{2}$	9.8	$10\frac{1}{4}$	7.6	Cleaned, regreased, and cup leather renewed before being tested.
			12	...	...	$20\frac{1}{2}$	7.5	$20\frac{1}{4}$	6.4	
			18	...	...	$30\frac{1}{2}$	6.8	$30\frac{1}{4}$	6.0	
			24	...	...	41	7.5	$40\frac{1}{2}$	6.4	
			30	...	...	51	7.1	$50\frac{1}{4}$	5.7	
B	4	12.56	6	$10\frac{5}{8}$	10.0	11	13.1	$10\frac{1}{2}$	9.0	Tested before and after renewal of cup leather.
			12	21	9.0	$21\frac{1}{2}$	11.1	$20\frac{1}{2}$	6.8	
			18	$31\frac{1}{8}$	7.9	32	10.4	$30\frac{5}{8}$	6.4	
			24	$41\frac{1}{4}$	7.3	42	9.0	$40\frac{5}{8}$	5.9	
			30	$51\frac{1}{4}$	6.8	52	8.1	$50\frac{5}{8}$	5.6	
				lb.		lb.				
C	$3\frac{1}{4}$	8.29	2	600	9.9	630	14.2	...	...	Ram comparatively new; packing in place of cup leather.
			4	1210	10.6	1230	12.1	...	...	
			$4\frac{1}{2}$	1350	9.9	1380	11.9	...	...	
			5	1500	9.9	1530	11.7	...	...	
				cwt.		cwt.		cwt.		
D	$8\frac{7}{16}$	55.91	10	4	10.6	4	10.6	4	10.6	Tested before and after renewal of cup leathers. Those removed, however, were in good condition.
			20	$7\frac{3}{4}$	7.7	8	10.6	$7\frac{3}{4}$	7.7	
			30	$11\frac{5}{8}$	7.7	12	10.6	$11\frac{5}{8}$	7.7	
			40	$15\frac{3}{8}$	7.1	16	10.6	$15\frac{3}{8}$	7.1	
			50	$19\frac{1}{8}$	6.5	$19\frac{3}{4}$	9.4	$19\frac{1}{8}$	6.5	
			60	23	6.7	$23\frac{7}{8}$	10.1	23	6.7	
				lb.		lb.		lb.		
E	$13\frac{5}{8}$	145.8	15	284	18.8	300	23.2	284	18.8	Ditto. ditto.
			30	520	11.4	545	15.4	520	11.4	
			45	760	9.0	790	12.5	760	9.0	
			60	1000	7.7	1030	10.5	1000	7.7	
F	$14\frac{3}{16}$	158.09	15	240	11.4	250	15.0	...	...	This ram is used for straightening purposes as a hydraulic bear; it is fixed in a horizontal position. It was formerly fitted with cup leathers, but as these seldom lasted more than a week, they were replaced by packing which has not been renewed for twelve months. The bear has been in constant use night and day.
			30	450	5.5	470	9.6	...	...	
			45	670	4.8	690	7.6	...	...	
			60	880	3.4	920	7.6	...	...	

Notes on Foundations.

Structure.	Nature of Foundation.	Load on Foundation.	Depth from H.-W. to River Bed.	River Bed to Bottom of Foundation.	Construction of Piers.	Load on Concrete or Masonry Filling.	Notes.
		tons per square foot.	feet	feet		tons per sq. ft.	
Forth Bridge, Queensferry Pier ...	Boulder clay	Dead and live load 4¼; 56 lb. wind, 1½; total gross load, 5¾	25	74	Rubble concrete in cylinders 70-ft. diameter	9 to 12	Concrete was made as follows:— 27 cubic feet broken whinstone rock, 7 cubic feet sand, and 5½ cubic feet cement made 1 cubic yard of concrete, weighing 37 cwt. Crushing strength 50 tons per square foot; tensile (transverse) 10 to 12 tons per square foot.
New Tay Viaduct, Dundee... ..	Silty sand	3½ tons net	38	25	Cylinders filled with concrete	—	
New Tay Viaduct, trial cylinder ...	Do.	7 tons gross, 5¾ tons net	...	...	Cylinders filled with concrete	...	Settlement, 5¼ in.
New Tay Viaduct, test on piers of old bridge	Clean sand	3.6 tons gross, 2.87 tons net	...	...	Cylinders filled with concrete	...	Settlement, ¼ in.
New Tay Viaduct, do. do. ...	Fine micaceous sand	3.6 tons gross, 2.87 tons net	...	...	Cylinders filled with concrete	...	Settlement, 2 in.
Tower Bridge, London	London clay (Boulder) ..	4 tons gross	34	26	Caissons filled with concrete	...	Load and settlement during building of structure. 1.3 tons per sq. ft., ⅜ in. 2.4 do. ⅞ in. and 1⅛ in. 3.1 do. 2 in. 3.2 do. 2⅜ in. 4.0 do. 3 in.
Tower Bridge, trial cylinder ...	London clay (Boulder) ..	4 tons gross	...	...	Caissons filled with concrete	...	Settlement after two days, 1½ in.
Alexander III. Bridge, Paris ...	Bed of compact flinty chalk ..	1.83 tons	...	...	Caissons filled with concrete	—	
Abbas II. Bridge, Cairo, Egypt ...	Coarse grey sand	5.4 tons gross	40	55	Cylinders (18-ft. diameter) filled with concrete	9¼	No settlement.

Blackfriars Railway Bridge, London (Arch spans)	London clay (Boulder)	$4\frac{3}{4}$ tons gross	30	20	Rectangular caisson filled with concrete	...	Settled $1\frac{1}{4}$ in.
Charing Cross Railway Bridge, London	Do. do.	8 tons gross	about 20	35	Cylinders filled with concrete	...	Settlement, about 1 in.
Campanile Tower, Cremona ...	Gravel ...	12 tons gross	...	...	Concrete and rubble	...	
Caledonian Railway, Viaduct over River Clyde at Glasgow, 1880	Sand, clay, and mud, with a few beds of gravel	11.6 tons gross	30	65	Cylinder filled with concrete	...	Sunk by open grabbing.
Glasgow Bridge (Road), 1899 ...	Bed of coarse sand and gravel of considerable thickness at 75 ft. in depth; also on rock at 106 ft.	6.5 tons gross	14	60	Concrete in cylinders 15-ft. diameter	$6\frac{1}{2}$	
Glasgow Bridge (Road), trial cylinder	Sand and gravel	7.7 tons gross	...	...	...	...	After 15 days there was no settlement.
Hawkesbury Bridge, Australia ..	... ...	12 tons gross	54	108	Caissons filled with concrete	6	If 300 lb. per square foot be deducted for skin friction, the pressure on the ground is 9 tons per square foot.
Clifton Arch Bridge, America ...	Gravel and boulders	$2\frac{1}{4}$ tons gross	6	...	Concrete	—	
Dufferin Bridge, Benares	Clay	11.19 tons gross	55	85	Concrete in cylinder	...	3 in. settlement. If displacement be allowed for, the pressure on the foundation is about one half.
Great Central Railway Warehouses at Marylebone, London	London clay (Boulder) ...	3 tons gross	...	...	Concrete blocks	—	
150-ton Cantilever Crane at Clydebank	Boulder clay and gravel ..	6 tons gross	15	58	Concrete in cylinder $13\frac{1}{2}$-ft. diameter	10	No settlement.
London Bridge	Boulder clay	5 tons gross, 60 to 80 tons per pile	...	...	Concrete on piled foundation	...	Piles 12 in. by 14 in. and from $3\frac{1}{2}$ ft. to $4\frac{1}{2}$ ft. centres.
Westminster Bridge	Do.	2 tons gross, 12 tons per pile		...	Concrete on piled foundation	—	
Iron Coal Pier at Norfolk, Va. ...	Sand	5 tons gross	...	...	Disc piles 4-ft. diameter	...	Sunk by water jet. When tested with 64 tons per pile, the average set was 13 in., minimum $\frac{1}{2}$ in., maximum 6 ft.
Swing Bridge over Avon, Bristol ...	Red Marl	2.9 tons gross	27	18	Concrete 7 to 1 with displacers	...	Piled foundations, load per pile 25 tons. All piles driven by 1-ton monkey falling 10 ft. until total set for the last five blows did not exceed 1 in.
150-ton Crane, Wallsend	Boulder clay	6 tons gross	8	60	Concrete 4—2—1	10	

Notes on Foundations.

THE approximate safe load in tons per square foot is given in the following Table, and will serve as a guide in preliminary designs. In foundations for important works it is recommended that bores or trial pits be sunk, and the bearing capacity of the soil ascertained before starting the final designs. All the values given are for foundations at depths below weather influences, and no allowance has been made for the weight of displaced soil, buoyancy, or the friction between the ground and cylinders or caissons.

Description of Ground.	Approximate Safe Load in Tons per Square Foot.
Bog, morass, quicksand, peat moss, marsh land Mud, hard peat turf, silt	0 to $\frac{1}{4}$
Soft, wet, or muddy clays and alluvial deposits of moderate depth in river beds	$\frac{1}{4}$ to $\frac{1}{2}$
Diluvial clay beds of rivers	$\frac{1}{2}$ to 1
Soft clay and wet sand	1
Alluvial earth, loams, and loamy soil (clay with 40 to 70 per cent. of sand), and clay loams (clay with about 30 per cent. of sand)	$\frac{3}{4}$ to $1\frac{1}{2}$
Ordinary clay and dry sand mixed with clay	2
Loose sand in shifting river beds, the safe load increasing with the depth	$2\frac{1}{2}$ to 3
Dry sand and dry clay	3
Silty sand of uniform and firm character in a river bed, secure from scour, and at depth greater than 25 ft.	$3\frac{1}{2}$ to 4
Hard clay mixed with very coarse sand	4
Sound yellow clay, containing only the normal quantity of water	4 to 6
Solid blue clay, marl and indurated marl, and firm boulder gravel and sand	5 to 8
Soft chalk, impure and argillaceous	1 to $1\frac{1}{2}$
Hard white chalk	$2\frac{1}{2}$ to 4
Ordinary superficial sand beds Firm sand in estuaries, bays, etc.	$4\frac{1}{2}$ to 5
Firm compact sand and gravel foundations at a depth not less than 20 ft.	6
Firm shale, protected from the weather, and clean gravel	6 to 8
Compact gravel	7 to 9

The above Table is taken principally from "Cylinder Bridge Piers," by J. Newman. Published by E. and F. N. Spon, Limited.

In proportioning the area of foundations, the pressure upon the ground should be kept as nearly as possible the same, so that the settlement (if any) may be uniform. In the case of buildings, such as warehouses, hotels, etc., the foundations of walls and piers should be proportioned for the dead load only, and be of such an area that the additional pressure due to the live load does not exceed the safe pressure on the ground.

Friction.

The support afforded to cylinder or caisson foundation by surface friction will depend upon the nature of the strata and the depth the cylinder or caisson has been sunk into the ground.

In practice it is not usual to rely upon friction as a supporting force, especially where the cylinders or caissons are increased in diameter at the bottom.

The average frictional resistance to drawing three hundred ordinary rough Memel timber piles, which had been driven into clay, was found to be about $16\frac{3}{4}$ cwt. per square foot of pile.[1]

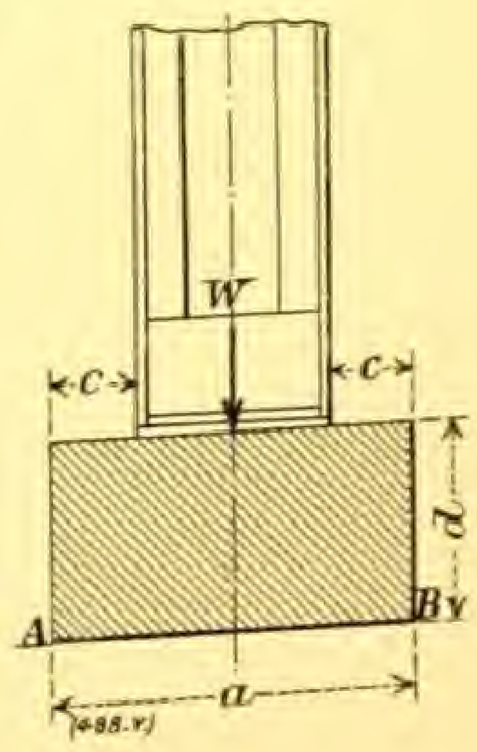
The following Table gives the approximate surface friction that probably occurs in various cases :—

Description of Ground and Material in Contact.	Approximate Surface Friction in Pounds per Square Foot.
Mud and silt on dry timber sawn piles - - - - - -	100 to 150
Soft clay on timber sawn piles - - - - - - - -	160 to 180
Sharp sand on clean timber sawn piles - - - - - -	1100 to 1500
Fine soft drift sand do. do. - - - - - - -	1500 to 1700
Mud and silt on clean, unplaned cast iron - - - - -	50 to 70
Sandy mud do. do. - - - - - -	150 to 250
Muddy clay do. do. - - - - - -	250 to 400
Ordinary sand on unplaned cast iron - - - - - -	300 to 400
Clean river-bed sand and gravel on unplaned cast iron - - -	400 to 600
Hard compact clay on unplaned cast iron - - - - - -	900 to 1000
Ordinary clay beds do. do. - - - - - -	700 to 800
Silty fine sand, liquid when disturbed by water, on unplaned cast iron	250 to 300

The clays above referred to are supposed to contain the normal quantity of water.

Concrete Foundations.

The necessary thickness of concrete foundations that they may distribute a load W uniformly over the surface A B may be found as above.



Using the same notation, we have

$$d = 1.73\, c \sqrt{\frac{W}{f\, b\, a}}.$$

The ultimate tensile resistance in a beam of good cement or lias-lime concrete is about 100 lb. per square inch, and in a beam of good brickwork in cement as much sometimes as 350 lb. per square inch.

With a pressure upon the surface A B of, say 3 tons per square foot, and a factor of safety of 2,

$$d = 1.73\, c.$$

[1] "Minutes of Proceedings of the Institution of Civil Engineers," vol. lxiv., page 313.

Timber Piles.

Let W = Safe load on pile in tons.
P = Weight of hammer in tons.
h = Distance of free fall of hammer in feet.
s = Penetration of pile for the last blow in inches.

Then $$W = \frac{2\,Ph}{s+1}.$$

This formula is supposed to give a factor of safety of about 6. If a pile is driven with a hammer weighing a ton falling 20 ft. and causing a penetration, say, of 1 in. for the last blow, W would be 20 tons, which in ordinary practice is the usual load allowed on piles well driven.

Resistance of Piles in Sand.

MR. J. SANDEMAN carried out a series of experiments to ascertain the resistance to horizontal stress of piles driven in different materials. The results showed that the greatest resistance was offered by sand, clay less, and loose ashes the least. All the piles broke off about 5 ft. below the surface of the ground. These experiments are described in vol. xli. of the "Proceedings of the Institution of Civil Engineers."

Weight of Timber Wet and Dry.[1]

IN computing the weight of timber in caissons and similar work, it is necessary that it should be calculated at its saturated weight. The following Table gives the weight of greenheart, elm, and oak when dry and after immersion in water:—

—	Weight per Cubic Foot.		Percentage Increase in Weight.
	lb.	oz.	
Greenheart Timber.			
When dry - - - - - - - - -	71	12	—
After 7 days in water - - - - - -	73	5	2.18
After 1 month in water - - - - -	74	11	3.57
After 2 months in water - - - - -	75	5	4.97
American Elm.			
When very dry - - - - - - - -	57	5	—
After 7 days in water - - - - - -	60	14	6.22
After 1 month in water - - - - -	63	12	11.23
After 2 months in water - - - - -	65	10	14.51
Dantzic Oak			
When very dry - - - - - - - -	39	0	—
After 10 days in water - - - - -	53	0	35.9

[1] Compiled from "Notes on Construction in Mild Steel," by H. Fidler.

Timber.

THE results of the tests given in the following Tables are of special value, as they were obtained from experiments on large timbers, such as are generally used in practice.

Transverse, Compressive, and Tensile Tests on Beams of Yellow Pine.[1]

Span of Beam.	Depth and Width.	Weight per Cubic Foot	Modulus of Elasticity.	Transverse Strength.	Compressive Strength.	Tensile Strength.
ft.	in.	lbs.	tons per square inch	tons per square inch	tons per square inch	tons per square inch
10	6 × 12	37	412	1.90	1.18	1.78
10	12 × 6	36	..	1.62	1.06	1.73
14	12 × 12	40.6	443	1.82	—	—
14	9 × 9	32.3	395	1.71	—	—
14	18 × 9	31.6	...	1.57	—	—

The results given in the above Table are in all cases the average of three tests. The timber used was Quebec yellow pine of good quality, which had been lying in a yard for five months after being purchased from the docks. It had not been seasoned, and when tested contained about as much moisture as most timber when actually used. The baulks were sawn, and to clearly show the fractures, all except those measuring 12 in. by 12 in. were passed through the planing machine.

Compressive Tests on Fir or Pine Timbers.[2]

All Timbers 12 in. by 12 in. in section.	Ratio of Length to Width.								
	10	15	20	25	30	35	40	45	50
Breaking Weight in Compression: Tons per square foot of section -	120	118	115	100	90	84	80	77	75

For various methods of preserving timber, creosoting, etc., see "Notes on Docks and Dock Construction," by C. Colson, M.I.C.E.

[1] "Minutes of Proceedings of the Institution of Civil Engineers," vol. cxxviii., page 334.

[2] *Ibid.*, vol. xxix., page 66.

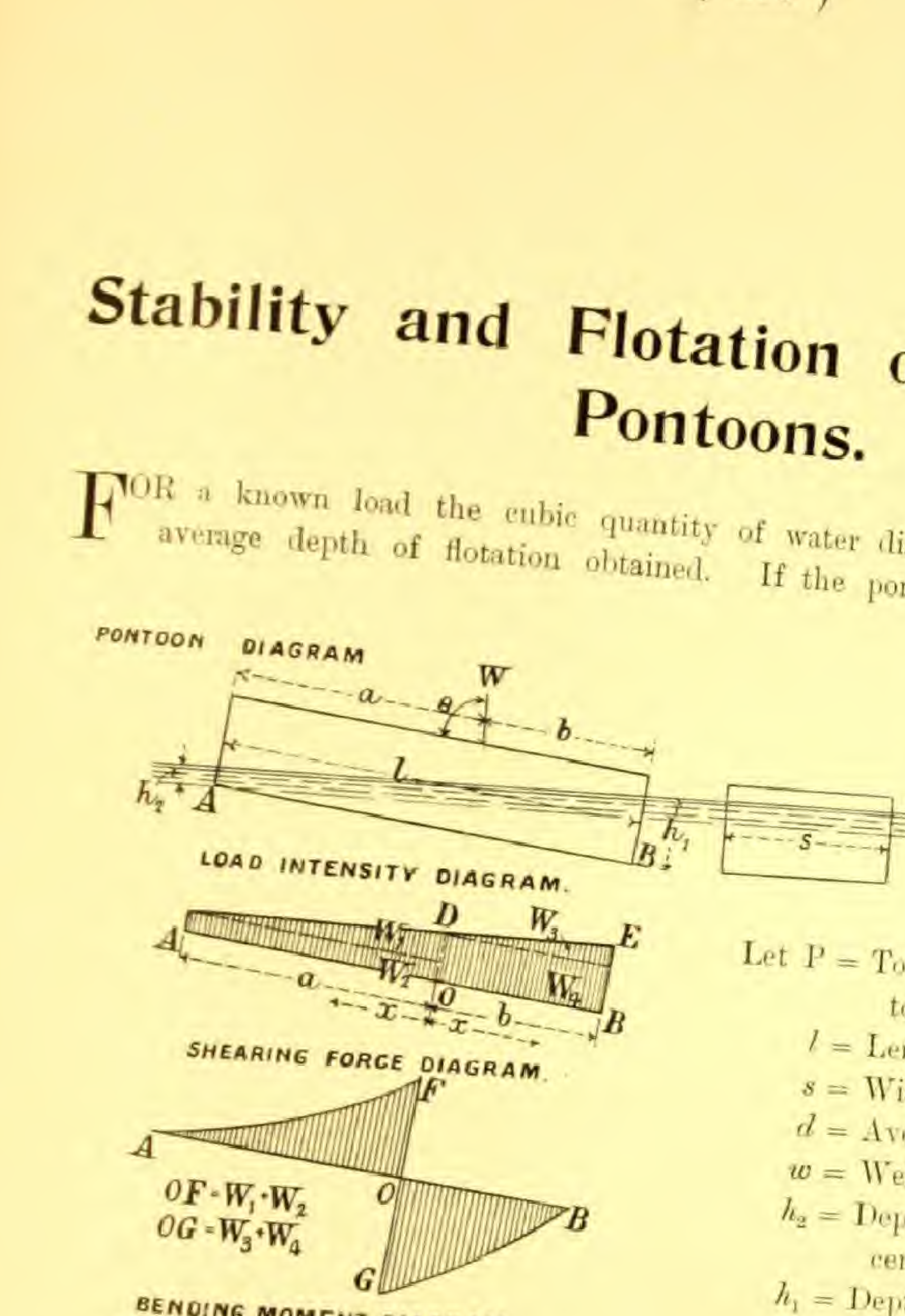
AVERAGE SAFE UNIT WORKING STRESSES FOR TIMBER IN POUNDS PER SQUARE INCH.

Kind of Timber.	Tension.		Compression.			Transverse.		Shearing.		Weight per Cubic Foot in Pounds.
	With Grain.	Across Grain.	With Grain. End Bearing.	With Grain. Columns under 15 Diameters.	Across Grain.	Extreme Fibre Stresses.	Modulus of Elasticity.	With Grain.	Across Grain.	
Factor of Safety	Ten	Ten	Five	Five	Four	Six	Two	Four	Four	
White Oak (Quercus alba)	1000	200	1400	900	500	1000	550,000	200	1000	49.94
White Pine (Pinus strobus)	700	50	1100	700	200	700	500,000	100	500	23.72
Southern Long-leaf or Georgia Yellow Pine (Pinus palustris)	1200	60	1600	1000	350	1200	850,000	150	1250	38.08
Douglas (Pseudotsuga douglasii), Oregon or Yellow Fir	1200	...	1600	1200	300	1100	700,000	150	...	31.84
Washington Fir or Pine (Red Fir)	1000	...	...	...	...	800	...	...	...	28.84
Northern or Short-leaf Yellow Pine (Pinus echinata)	900	50	1200	800	250	1000	600,000	100	1000	31.84
Red Pine	900	50	1200	800	200	800	600,000	...	...	31.21
Norway Pine (Pinus resinosa)	800	...	1200	800	200	700	600,000	...	...	31.21
Canadian (Ottawa) White Pine	1000	...	...	1000	...	...	...	100	...	37.46
Canadian (Ontario) Red Pine	1000	...	...	1000	...	800	700,000	100	...	34.34
Spruce and Eastern Fir	800	50	1200	800	200	700	600,000	100	750	24.97
Hemlock	600	...	...	800	150	600	450,000	100	600	24.97
Cypress (Taxodium distichum)	600	...	1200	800	200	800	450,000	...	...	28.72
Cedar	800	...	1200	800	200	800	350,000	...	400	23.10
Chestnut	900	...	...	1000	250	800	500,000	150	400	41.20
California Redwood	700	...	...	800	200	750	350,000	100	...	24.16
California Spruce	...	...	...	800	...	800	600,000	...	...	24.97

Recommended by the Committee on "Strength of Bridge and Trestle Timbers" of the Association of Railway Superintendents of Bridges and Buildings, October, 1895. The weights are for wood containing 15 per cent. of moisture, and known as "dry timber." Green and unseasoned timber will be from 20 to 40 per cent. heavier.

Stability and Flotation of Rectangular Pontoons.

FOR a known load the cubic quantity of water displaced is determined, and the average depth of flotation obtained. If the pontoon is loaded symmetrically about both longitudinal and transverse axes, the depth of flotation is uniform; but if it is loaded unsymmetrically about one axis the depth of flotation at either end of this axis may be obtained as follows:—



Let P = Total downward force on pontoon, including its own weight.

l = Length of pontoon = $(a + b)$

s = Width of pontoon.

d = Average depth of flotation.

w = Weight per cubic foot of water.

h_2 = Depth of flotation at end nearer centre of gravity.

h_1 = Depth of flotation at end further from centre of gravity.

b = Shorter distance from centre of gravity to end of pontoon.

Then $d = \dfrac{P}{wsl} = \dfrac{h_1 + h_2}{2}$ $\qquad b = \dfrac{h_2 + 2h_1}{h_2 + h_1} \cdot \dfrac{l}{3}$

And $h_1 = \dfrac{6\,db}{l} - 2\,d$ $\qquad h_2 = 2\,d - h_1$

very approximately.

The centre of gravity of the loads G should intersect the centre of gravity of the displaced water.

Case for Concentrated Load W *at any Point, as in figure.*

Loading: $\quad W_1 = \dfrac{a^2 (h_2 - h_1)}{l^2 (h_2 + h_1)} W \sin\theta \qquad W_2 = \dfrac{2\,ah_1}{l\,(h_2 + h_1)} W \sin\theta$

$$W_3 = \frac{b^2 (h_2 - h_1)}{l^2 (h_2 + h_1)} W \sin\theta \qquad W_4 = \frac{2\,b(h_2 a + h_1 b)}{l^2 (h_2 + h_1)} W \sin\theta.$$

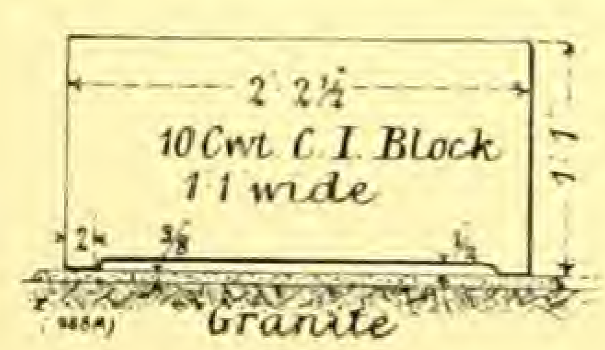
Origin at O.

—	Left of O.	Right of O.
Load intensity	$\frac{2\,W_1}{a}\left(a - x\right) + \frac{W_2}{a}$	$\frac{2\,W_3}{b}x + \frac{W_4}{b}$
Shearing force	$-\frac{W_1}{a^2}\left(a - x\right)^2 - \frac{W_2}{a}\left(a - x\right)$	$-\frac{W_3}{b^2}\left(b^2 - x^2\right) - \frac{W_4}{b}\left(b - x\right)$
Bending moment	$\frac{W_1}{3a^2}\left(a - x\right)^3 + \frac{W_2}{2a}\left(a - x\right)^2$	$\frac{W_3}{3\,b^2}\left(2\,b^3 - 3\,b^2\,x + x^3\right) + \frac{W_4}{2\,b}\left(b - x\right)^2$
Moment at O, M_o	$\frac{W_1 a}{3} + \frac{W_2 a}{2}$ =	$\frac{2\,W_3\,b}{3} + \frac{W_4 b}{2}$
Shear at O -	$W_1 + W_2$	$W_3 + W_4$
Load intensity at O	$\frac{2\,W_1}{a} + \frac{W_2}{a}$ =	$\frac{W_4}{b}$
Load intensity	$\frac{W_2}{a}$ at A	$\frac{W_4}{b}$ at B

For a series of loads the above treatment may be applied separately to each and the algebraic sum taken.

Shearing Strength of Mortar Joint.

A 10-cwt. cast-iron block was laid frog downwards on a bed of cement mortar (2 to 1), the mortar being spread on the top of a granite course. It was allowed to set for five days, and then the block was slid horizontally by means of an hydraulic jack. The pressure required to destroy the adhesion of the mortar to the granite (which was the line of failure) was 4 tons per square foot of bed, equal to 63 lb. per square inch.



Strength of Brickwork.

THE following values are compiled from experiments made in 1895 by the Science Standing Committee of the Royal Institute of British Architects.[1] Piers were built of different varieties of brick, 6 ft. high by 18 in. square. Two of each were built in lime mortar and two in Portland cement mortar. The tests were made at the end of three months and six months after building. The brickwork was as nearly

[1] "Report on Brickwork Tests," published by the Institute.

as possible four courses to the foot, and the bond was that shown in Rivington's text book, being the ordinary way of building an 18 in. pier. The cement mortar consisted of 1 part of Portland cement to 4 parts of Thames river sand, washed and sifted. The lime mortar was of 1 part grey stone lime, not ground and well slaked, to 2 parts of washed river sand. Stock bricks and Fletton bricks had frogs of varying dimensions. Red Ellistown bricks had no frogs. Fletton bricks were pressed machine made, not wire cut. Staffordshire blue bricks had no frogs, and were wire cut, but not pressed.

Report on Crushing Tests of Samples of Bricks used in the Experimental Piers.

Description.	Area Crushed in Square Feet (Average).	Mean Crushing Strength in Tons per Square Foot.
London stocks	.261	84.27
Gault	.258	182.2
Leicester red (half brick)	.130	382.1
Blue Stafford (half brick)	.125	701.1
London stocks (whole)	.272	92.85
" (half)	.140	46.94
Red rubbers (whole)	.337	84.11
" (half)	.165	58.71
Gaults (whole)	.253	192.2
" (half)	.134	160.7

Excluding London stocks which are irregular in strength and cannot be divided neatly, it appears that bricks tested as half bricks may be expected to crush with about 25 per cent. less pressure per square foot than the same bricks tested as whole bricks.

Tests.

Description.	Age in Weeks.	Crushing Strength in Tons per Square Foot (Mean).
Grey Lime Briquettes. 2 parts standard (Leighton Buzzard) sand to 1 of lime by volume.	4	6.08
	12	8.73
	24	15.72
	34	28.19
Portland Cement Briquettes. 4 parts standard sand to 1 of cement by volume.	4	31.45
	13.7	48.52
	24	56.15
	36	58.62
Do. do. But with special sand as used in building the piers.	24	29.00

TESTS OF BRICK PIERS. AVERAGE VALUES.

1. *Brick only.*

Name of Brick.	Where from.	Commencement of Failure in Tons per Foot Superficial.	Total Failure in Tons per Foot Superficial.
London stocks -	Sittingbourne -	76.25	84.27
Gault - - -	Burham - -	102.3	182.2
Leicester red - -	Ellistown - -	248.15	382.1
Staffordshire blue -	Rowley Regis -	471.6	701.0
Fletton - - -	Fletton district near Peterborough -	...	220.85

2. *Brickwork in Mortar.*

Name of Brick.	Where From.	Age of Brickwork in Weeks.	Commencement of Failure in Tons per Foot Superficial.	Total Failure in Tons per Foot Superficial.
London stocks -	Sittingbourne -	$18\frac{2}{7}$*	4.18	10.41
		$43\frac{4}{7}$	7.49	12.54
		$22\frac{4}{7}$	14.25	18.64
Gault - - -	Burham - -	$18\frac{4}{7}$	5.58	21.92
		$43\frac{6}{7}$	9.40	21.60
		$22\frac{4}{7}$	15.44	31.14
Leicester red - -	Ellistown -	$18\frac{6}{7}$	15.65	30.74
		$43\frac{6}{7}$	14.11	34.12
		$21\frac{6}{7}$	28.14	45.36
Staffordshire blue -	Rowley Regis -	19	28.53	74.30
		$44\frac{2}{7}$	19.49	73.66
		$21\frac{6}{7}$	24.08	114.34
Fletton - - -	Fletton district	$22\frac{1}{7}$	24.08	30.68
3. *Brickwork in Cement.*				
London stocks -	Sittingbourne -	21	10.28	14.93
		$45\frac{6}{7}$	9.05	16.96
		$21\frac{5}{7}$	27.45	39.29
Gault - - -	Burham - -	$20\frac{5}{7}$*	12.09	17.79
		$45\frac{6}{7}$	18.72	29.98
		22	37.24	51.34
Leicester red - -	Ellistown -	$22\frac{3}{7}$*	22.82	58.45
		46	23.13	50.43
		21	62.33	83.36
Staffordshire blue -	Rowley Regis -	$22\frac{2}{7}$*	31.6	72.80
		$46\frac{4}{7}$	36.86	82.48
		$21\frac{3}{7}$	84.99	135.43
Fletton - - -	Fletton district	$20\frac{5}{7}$	43.88	56.25

* Only in the case of those marked with an asterisk, a portion of the pier was filled in with closers of London stocks, and the results are given here because they represent probable conditions.

Holding Power of Bolts in Masonry.

Bolts Ragged for $3\frac{1}{8}$ in. at End and Fixed into Limestone.

Adhesion in Cwts. per Square Inch.	Diameter of Hole in Masonry.	Diameter of Bolt.	Length of Bolt in Masonry.	Material Run in Hole.	Test.	Remarks.	
1.45	$1\frac{3}{8}$ in.	$\frac{3}{4}$ in.	3 ft. 6 in.	Sulphur	16,000 lb.	Bolt broke	2 weeks old
"	"	"	"	"	16,000 "	"	"
"	"	"	"	Lead	16,000 "	"	"
"	"	"	"	"	16,000 "	"	"
"	"	"	"	Cement neat	16,000 "	"	"
"	"	"	"	"	16,000 "	"	"
"	"	"	"	"	16,000 "	"	"
2.10	"	1 in.	"	Sulphur	31,000 "	"	"
0.81	"	"	"	"	12,000 "	Bolt drew out	"
2.10	"	"	"	Lead	31,000 "	Bolt broke	"
0.88	"	"	"	"	13,000 "	Bolt drew out	"
2.10	"	"	"	Cement neat	31,000 "	Bolt broke	"
2.10	"	"	"	"	31,000 "	"	"
1.76	"	"	"	"	26,000 "	Bolt drew out and broke	"

Bolts Fixed in Limestone Blocks 18 in. × 12 in. × 10 in.

4.73	$1\frac{3}{4}$ in.	1 in. plain	12 in.	Cement	20,000 lb.	Cement began to yield	13 days old
4.98	"	1 in. screwed	"	"	21,000 "	"	"
4.02	$2\frac{3}{4}$ in.	2 in. plain	"	"	34,000 "	"	"
7.94	"	2 in. screwed	"	"	67,000 "	Stones split	"
3.79	"	2 in. plain	"	"	32,000 "	Bolts began to yield	"
5.92	"	2 in. screwed	"	"	50,000 "	Stones split	"

"Scientific American," vol. lxiii.

Cement and Concrete.

Compressive Strength of Concrete.[1] (In tons per square foot after about a year.)

Lime or Cement.	Proportion of Lime or Cement to Sand and Gravel.			
	1 to 6	1 to 8	1 to 10	1 to 12
Grey lime	10.2	4.6	5.2	—
Lias lime	11.4	11.1	11.5	—
Aberthaw lime	34.1	21.8	15.4	—
Portland cement	100.7	76.4	53.5	37.1

[1] "The Principles of Structural Design." Scott-Moncrieff.

Transverse Strength of Concrete.[1]

No.	Composition.			Age in Days.	Modulus of Rupture in Pounds per Square Inch.	Remarks.
	Portland Cement.	Sand.	Aggregate.			
1	1	...	1 Coke breeze	7	672	A comparison of the results of these experiments, especially Nos. 7, 8, and 9, shows that not only the amount of cementitious material, but the graduating of the sizes of the aggregate, so as to fill the interstices, is of importance.
2	1	...	2 Crushed brick	6	195	
3	1	0	5 Shingle	139	330	
4	1	1	5 ,,	139	247	
5	1	2	5 ,,	139	159	
6	1	3	5 ,,	139	99	
7	1	2	2 Broken stone, 1½" / 4 ,, ,, 3½"	90	174	
8	1	2	6 Gravel	90	156	
9	1	2	6 Broken stone, 1½"	90	136	
10	1	0	9 Shingle	95	142	
11	1	1	8 ,,	95	132	
12	1	2	7 ,,	95	105	

The following Table shows the quantity of Portland cement required to make 1 cubic yard of concrete, cement weighing 1 cwt. per bushel:—

Cement.

11 bags = 1 ton.

1 bag = 203 lb.

Proportions.	Cement.		
	cwt.	qrs.	lbs.
1 cement to 1 ballast.	13	0	0
1 ,, 2 ,,	8	2	0
1 ,, 3 ,,	6	2	0
1 ,, 4 ,,	5	1	0
1 ,, 5 ,,	4	1	0
1 ,, 6 ,,	3	3	0
1 ,, 7 ,,	3	1	0
1 ,, 8 ,,	3	0	0
1 ,, 9 ,,	2	2	14
1 ,, 10 ,,	2	2	0

For 6 to 1 Portland Cement Concrete.

1 cubic foot = 136 lb.

1 ,, yard = 1.64 tons.

16½ ,, feet = 1 ton.

2 bags for 1 cubic yard of concrete = 3 cwts. 2 qrs. 14 lb.

The ultimate tensile resistance in a beam of good cement or lias lime concrete is about 100 lb. per square inch.[2]

[1] "The Principles of Structural Design." Scott-Moncrieff.

[2] Baker, "Lateral Pressure of Earthwork." (M.P.I.C.E., vol. lxv.)

STRENGTH OF CONCRETE.

The following results in Table I. are from experiments on Portland cement concrete cubes by Mr. G. F. Deacon in the construction of the Vyrnwy Dam. The second Table is given by Trautwine for Portland cement concrete. The third series of results are from the "Minutes of Proceedings of the Institution of Civil Engineers," vol. xxv., but the values there given are reduced to tons per square foot.

Table Number.	Composition.	Age of Block in Months.	Crushing Strength in Tons per Square Foot.
1	—	1 — 2 2 — 5 17 —$25\frac{1}{2}$ $28\frac{1}{2}$—$29\frac{1}{2}$ 32 —36	Over 114 „ 102 „ 159 „ 162 „ 180
2	1 to 5	1 3 6 9 12	15 (mean) 40 „ 65 „ 85 „ 100 „

3	Number of Months Made.		
	Three.	Six.	Nine.
Neat Portland cement	244	343	385
1 cement to 1 sand	160	223	293
1 „ 2 „	129	177	235
1 „ 3 „	92	139	154
1 „ 4 „	86	115	142
1 „ 5 „	62	99	108

About five-eights of the crushing weights produced the first crack.

The sand to be used with Portland cement should be clean, sharp, angular and free from earthy substance. Cold water should be used for mixing, the quantity being just sufficient to cause it to become of the consistency of stiff mortar. Salt water is equally as good as fresh, except for stucco work. The bricks or work for which Portland cement is used should be first well wetted, and when the cement has commenced to set, the process should never be disturbed, as it cannot be renewed. When the work has set it will improve by being immersed in water. Cement blocks kept under water are stronger than those exposed to the atmosphere. Bricks of neat Portland cement are equal in strength to blue bricks, after they have been made a few months. The various tests on quality, etc., are given in the British Standards Specification for Portland Cement. Bitumen concrete, of dried coke breeze grouted with hot pitch and oil, weighs only 89 lb. per cubic foot.

Lime Mortar.

Crushing Strength of Lime Mortar 18 Months Old.		Tensile Strength of Lime Mortar.	
Description of Mortar.	Crushing Strength in Tons per Square Foot.	Description of Mortar.	Tensile Strength in Tons per Square Foot.
Lime mortar and river sand	28.0	Mortar of sand and hydraulic lime, well made	8.74
Lime mortar and pit sand	37.2	Mortar of sand and ordinary hydraulic lime, well made	5.46
Mortar made with pounded sandstone	26.8	Mortar of sand and ordinary lime, well made	3.28

The safe working stress in compression on ordinary lime mortar is 4 tons per square foot, and the average weight is 112 lb. per cubic foot.

Tensile Stress Required to Separate Bricks. Cemented together in Blocks of Four with Portland Cement and Lime Mortar. At the end of Twelve Months; all set in Air.

Description of Brick.	Area of Bed Separated in Square Inches.	Tensile Stress in Tons per Square Inch.						
		Portland Cement and Sand.						Blue Lias Lime and Sand.
		Neat.	1 to 1.	2 to 1.	3 to 1.	4 to 1.	5 to 1.	1 to 2.
Gaulty clay, pressed	36.1	11.3	11	6	6.9	5.1	...	10
„ wire cut	37.0	17	10.8	...	7.2	5.5	5.4	—
„ perforated	39.4	27.1	20.7	15.5	11.6	7.5	5.6	12.2
Suffolk	40.5	22.8	16.3	15.6	14.1	13.4	9.3	16.5
Stock	37.0	19.6	15.7	10.9	10.5	5.7	5.5	8.9
Fareham red	36.1	31.5	20.8	14.3	10.4	8.6	7.2	13.3
Staffordshire blue, pressed with frog	36.1	17.8	14	12.3	10.7	8.2	5.2	7.6
Staffordshire blue, rough, without frog	36.1	12.1	11.8	12.1	9.0	6.9	...	8.8

Durability of Iron in Water.

THE durability of wrought or cast iron in water is dependent on the impurities contained in the water, on the nature of the iron, and to some extent on the protection afforded by marine growths covering the surface. The portions liable to the greatest corrosion are those parts which are alternately wet and dry. The available records of durability are conflicting, but the general experience is that wrought iron is more durable in salt water than cast iron, and the opposite in fresh water. Steel is less durable than either in salt or fresh water.

Weight of Kentiledge for Counterweight.

THE stowage capacity of various materials used for counterweight or ballast in swing bridges, cranes, caissons, etc., is required to estimate the dimensions of the ballast chamber or box. In swing bridges where timber decking is used, the saturated weight of the timber from rain or snow should be taken in estimating the amount of counterweight necessary. In all cases an excess allowance of not less than 10 per cent. should be made in the capacity of the ballast box to allow a margin for adjustments and variations.

The following Table has been compiled from various sources to show the stowage value of different materials:—

Item.	Description of Ballast.	Weight per Cubic Foot in Pounds.	Cubic Feet per Ton.	Percentage of Interstices.
1	Cement concrete, composed of 4 parts of gravel, 1 of Portland cement, and 2 of sand.	135.0	16.6	Solid.
2	Concrete formed of nickel slag broken small, carefully packed, rammed in layers of about 1 ft. and grouted up solid with mortar composed of 1 part of Portland cement and 2 parts of sand.	167.0	13.42	Solid.
3	Rough broken cast iron in pieces easily handled, carefully packed in ballast box of swing bridge.	269.0	8.33	10
4	Rough pig iron, about 3 ft. 6 in. in length and of ordinary section, 4½ in. by 4 in., laid in rows in alternate directions, and stowed as close as possible.	284.05	7.88	37
5	The same pigs broken in short lengths of 12 in. and under, laid in rows in alternate directions, and stowed as close as possible.	287.22	7.79	36
6	Steel punchings (burrs) alone - - -	303.0	7.39	38
7	Same as No. 5, the interstices being filled up with steel burrs.	333.88	6.71	27
8	Steel burrs grouted with Portland cement mortar; 1 of cement to 1 of sand and rammed (burr concrete).	350.0	6.40	Practically solid.
9	Cast iron in blocks generally about 9 in. long by 3 in. square with shaped pieces of varying size, specially packed in a floating ship caisson and grouted up with cement mortar.	365.0	6.14	Practically solid.
10	Specially cast slabs to fit over stiffeners of plate girder for counterweight.	400	5.6	11

Aggregates in Concrete in Contact with Steel or Iron.

IN the widening of Blackfriars Bridge, London, the hoop iron bonding in the old abutments which were built forty years ago was laid bare. The abutment is 40 ft. thick, built of stock brickwork in cement mortar with a granite facework. The hoop iron in all parts of the abutment, both above and below high water, was badly corroded. The hoop iron, mortar, and brick were analysed. When the faces of the stock bricks were examined and the surface dissolved in distilled water, they gave 0.166 per cent. of sulphuric anhydrides and 0.C095 per cent. of chlorides. A surface of the hearting brick in contact with a corroded iron tie when extracted with distilled water gave 0.61 per cent. of sulphuric anhydrides and 0.005 per cent of chlorides. The corroded hoop iron removed from the brickwork gave 1.72 per cent. of sulphuric anhydrides and 0.06 per cent. of chlorides, both soluble in distilled water. The action of acid showed that only 4.7 per cent. of metallic iron was left in the corroded material. These analyses showed (1) that both the stock and hearting bricks contained salts which were soluble in water, notably among them being sulphates and a small proportion of chlorides; (2) that corrosion was almost complete in the hoop iron, and that acid radicles corresponding to those found in the bricks also occur in the iron; (3) that the formation of the iron salts was probably due to the action of the *salts present in the bricks*, and that in ferro-concrete construction the steel should be properly coated with cement and attention given to the aggregates used in the concrete.

Superimposed Loads on Floors of Buildings.

THE actual loads on the floors of three office buildings in Boston were investigated by Messrs. Blackall and Everett. The greatest number of people known to be in each office at any one time, in addition to the weight of furniture and the contents, were included in the loads. The greatest load in any office was found to be 40.2 lb. per square foot. The greatest average for all offices was found to be 17 lb. per square foot. In 12.4 per cent. of the offices the floor load was in excess of 25 lb. per square foot, and in 26 per cent. it exceeded 20 lb. per square foot. Mr. C. C. Schneider investigated the average floor loads in office buildings, and submitted his investigations in a Paper to the American Society of Civil Engineers in 1904. His investigations showed that floor beams should be designed to carry concentrated loads occupying any position on the beam as well as distributed load in excess of the average floor load. He cited the following examples of heavy local loading:—In an engineering office a number of cases with drawers holding drawings were placed in a double row, back to back, in the middle of a room. The cases were 31 in. wide and 36 in. high, weighing when completely filled 160 lb. per lineal foot, or both together 320 lb. per lineal foot, with a width of 72 in. A case of drawers for drawings 31 in. by 44 in. and 5 ft. high, if completely filled, would weigh 1200 lb., or 326 lb. per lineal foot. The weight of a "Wernicke" bookcase about 6½ ft. high was found to be 170 lb. per lineal foot when

completely filled with books. A row of these bookcases may be placed each side of a partition, and would therefore weigh 340 lb. per lineal foot in addition to the weight of the partition. Movable partitions may be assumed to weigh about 14 lb. per square foot. Any of these loads may run parallel to or across the floor beams. The weights of safes vary from about 10 cwt. upwards, and may be accepted as a minimum concentrated load. As special provision is made for the heavy safes, it is only necessary to consider the lighter ones as a movable load. The following live loads shall be assumed for the different classes of buildings and the maximum result used for determining the sections :—

(*a*). A uniform load per square foot of floor area ;

or (*b*). A concentrated load, occupying an area of 5 ft. by 5 ft., applied to any part of the floor ;

or (*c*). A uniform load per lineal foot for the floor beams.

Live Loads.

Class of Building.	A. Average Uniform Load per Square Foot.	B. Concentrated Load.	C. Load per Lineal Foot of Girder. — Not less than	Proportion of Depth of Beam to Span.
	cwts.	cwts.	cwts.	
Domestic dwellings, dormitories, infirmaries, and hospitals	$\frac{3}{4}$	10	5	$\frac{1}{25}$
Office buildings : Upper floors	$\frac{3}{4}$	20	5	$\frac{1}{25}$
Do. First floors	1	20	$7\frac{1}{2}$	$\frac{1}{25}$
Do. Showrooms, ground floor, stairs, and corridors	$1\frac{1}{4}$	25	10	$\frac{1}{20}$
Schoolrooms and theatres	$1\frac{1}{2}$	20	10	$\frac{1}{20}$
Public halls, assembly rooms, and drill halls	$1\frac{1}{2}$	30	10	$\frac{1}{20}$
Factories with light machine tools, ordinary stores, stables, and coach and motor houses	1	50	10	$\frac{1}{20}$
Warehouses and factories* / Power stations' uncovered floor*	from 2	from 50	from 15	$\frac{1}{20}$
Cell rooms for electric stations*	,, 4	,, 60	,, 15	$\frac{1}{15}$
Charging floors for foundries*	,, 3	,, 60	,, 20	$\frac{1}{15}$
Pavements in front of buildings	,, 2	,, 40	,, 15	$\frac{1}{15}$
Carriageways through buildings	,, 3	,, 80	15	$\frac{1}{15}$
Flat roofs	$\frac{1}{2}$	10	5	$\frac{1}{25}$
Do. used for storage	Not less than for upper floors			$\frac{1}{25}$

NOTE.—If the local bye-laws specify a uniform load per square foot, these loads must be taken instead of the loads under Column A. In the case of those marked with an asterisk, the actual loads are to be ascertained. The dead weight of a fireproof floor construction is to be taken at 56 lb. per square foot, and an ordinary timber floor at 28 lb. per square foot, in addition to the surface of the flooring.

Properties of Materials in Common Use.[1]

Material.	Average Weight Per Cubic Foot in Pounds.	Ultimate Strength in Tons per Square Inch. Tensile. Max.	Tensile. Min.	Compressive (small cube). Max.	Compressive (small cube). Min.	Shearing. With Grain.	Shearing. Across Grain.	Transverse. Max.	Transverse. Min.	Elastic Limit in Tons per Square Inch. Tension.	Elastic Limit Com-pression.	Modulus of Elasticity in Tons per Square Inch.
Timber.												
Ash	46 to 53	7.6	5.5	4.0	3.6	0.62	3.6	6.2	5.4	—	—	710
Beech	47 to 51	5.1	2.1	4.2	3.4	0.62	—	5.4	4.0	—	—	600
Elm, English	34 to 38	6.3	2.7	4.6	—	—	—	4.3	2.7	—	—	310 to 600
,, Canadian	45	—	—	—	—	—	—	—	—	—	—	—
Greenheart	71	—	—	—	—	—	—	—	—	—	—	—
Lignum Vitæ	41 to 73	—	—	—	—	—	—	—	—	—	—	—
Oak, English	58 to 61	6.0	4.0	5.0	2.0	1.0	2.0	6.1	4.5	—	—	540 to 780
,, American Red	53	Average	4.5	Average	2.7	.2 to .4	1.1 to 2.5	—	—	—	—	—
Pine, Red	36 to 41											—
,, North of England Fir	35	6.3	1.2	2.8	1.7	.22 to .36	2.2	4.3	2.2	—	—	640 to 850
,, Forest of Mar Fir	43											
,, White	27	—	—	—	—	.11 to .22	1.1	3.2	1.5	—	—	714
,, Yellow	32	Average	2.4	Average	2.4	—	—	—	—	—	—	690
,, Dantzic	40	—	2.4	Average	2.4	—	—	—	—	—	—	—
,, Memel	34	—	—	—	—	—	—	—	—	—	—	710
,, Pitch	42	5.4	2.1	3.0	2.2	—	—	6.3	—	—	—	560
,, Larch	34	6.3	4.0	Average	2.5	.43 to .76	—	4.5	2.2	—	—	400 to 610
,, Spruce	32	7.9	1.3	2.7	1.0	.27	—	5.5	1.6	—	—	630 to 803
Teak	47 to 61	Average	3.6	Average	5.4	—	—	—	—	—	—	1070 to 1250

Metals and Alloys.												
Aluminium, Wrought	167	10	7	—	—	—	—	—	—	2-7	8-10	2000 to 6000
,, Cast	160	—	—	—	—	—	—	—	—	—	—	—
Brass, Cast	525	Average	8.0	Average	4.7	—	—	—	—	2-4	—	4080
Copper, Hammered	556	Average	15.0	25	20	12	11	—	—	—	—	7000
,, Cast	549	11.5	8.5	—	—	—	—	—	—	—	—	6700 to 8030
Gunmetal	540	Average	8.5	Average	25.9	12	8	—	—	3-4	5-6	5000
Lead, Cast	710	Average	0.8	—	—	—	—	—	—	—	—	320
Manganese	499	—	—	—	—	—	—	—	—	—	—	—
Nickel, Hammered	541	—	—	—	—	—	—	—	—	—	—	—
,, Cast	518	—	—	—	—	—	—	—	—	—	—	—
Tin, Cast	462	Average	2.1	—	—	—	—	—	—	—	—	2050
Steel, Carbon, Mild	490	32	28	—	—	—	24-26	—	—	17-18	16-17	11,600 to 14,300
,, ,, Rivet	—	30	26	—	—	—	21-22	—	—	15-17	—	—
,, Nickel ($3\frac{1}{2}$ per cent.)	—	54	47	—	—	—	31	—	—	27	—	13,500
Iron, Cast	450	11	7	60	35	—	8 to 13	18	—	6.8	—	6,000 to 10,000
,, Wrought	480	24	21	Average	18	—	17 to 19	20	—	12 to 15	—	11,000 to 13,000
Zinc, Cast	428	Average	1.3	—	—	—	—	—	—	—	—	—
,, Sheet	449	—	—	—	—	—	—	—	—	—	—	—
Masonry, etc.												
Granite, Scotch	170	—	—	11.2	8.1	—	34 to 100	—	—	—	—	1340 to 4000
Sandstone, Scotch	151	—	—	5.4	0.94	—	12 to 68	0.97	0.56	—	—	1020 to 1350
,, English	—	—	—	4.38	2.34	—	—	—	—	—	—	—
Trap, Basalt	181	—	—	10.1	3.69	—	—	—	—	—	—	—
Limestone, Portland	168	—	—	3.58	1.17	—	40 to 64	1.7	0.67	—	—	680
,, Bath	—	—	—	0.65	0.54	—	—	—	—	—	—	—
Rubble Masonry	—	—	—	$\frac{1}{10}$ of	cut stone	—	—	—	—	—	—	—
Squared Stone (block in course)	—	—	—	1.39	1.04	—	—	—	—	—	—	—

[1] Compiled from various sources.

Weights of Materials and Merchandise.

Material.	Weight per Cubic Foot in Pounds.
Timbers.	
Mahogany, Spanish	53—66
,, Honduras	35
Plane	41
Poplar	25
,, White	32
,, 20 per cent. Water	30
Iron Bark	64
Black Butt	56
Cedar	35—37
Cypress	37
Maple	49
Cherry	42
Sycamore, dry	37
Karri	63
Masonry, Rock, etc.	
Rubble Masonry	115—144
Masonry, good rubble in mortar, well dressed	154
Masonry, good rubble in mortar, dry	138
Granite Masonry	160
Sandstone Masonry	140
Brickwork	112
Lime Mortar	109
,, hardened	103
Brick	125
Cement Concrete	130—150
Coke Breeze Concrete	89
Quick Lime	95
Portland Cement	81—102
Granite	160—170
,, Cornish	166
,, Aberdeen	164
Sandstone	130—157
Limestone, granular	125
,, compact	168
Trap Rock, Basalt	170—187
Quartz	165
Marble	164—172
Gneiss	168
Slate	162—180
Coal, ordinary broken, loose	56
,, anthracite	93.5
Coal, bituminous	77—90
,, ,, broken, loose	47—52
Coke	40—50
,, loose	23—32
Gypsum	142
Mica	183
Shales, red or black	162
Earths, Soils, etc.	
Earth, Common Loam, dry, loose	72—80
Earth, Common Loam, moist, loose	66—76
Mud, dry	80—110
,, wet	110—130
,, fluid	104—120
Sand, dry	90
,, damp	118
Gravel	109
Shale	162
Chalk	174
,, air dried	155
Clay	135
Miscellaneous.	
Water, 32 deg. Fahr.	62.4
Ice	57
Snow, fresh fallen	5—12
,, moistened and compacted	15—50
Asbestos	187
White Lead	197
Pitch	72
Tar	62—63½
Asphalt	88
Ballast	112
Cork	15
Plate glass	169
Window glass	154—157
Flint glass	187—208
Grain, loose	49
,, packed	54
Tallow	58
Petroleum	55
Salt	45
Sulphur	125

NOTE.—For additional weights, see page 414.

SPECIFICATIONS.

INTRODUCTION.

THE following specifications embody the general practice of Sir William Arrol and Company, Limited in the design and manufacture of cranes, bridges, workshop buildings, and general constructional steelwork. In their preparation the published specifications of British, Continental and American engineers have been consulted, and the specifications represent the best practice and principles in the design of first-class structures. The working stresses for railway bridge superstructures are based upon the specifications prepared by Sir Benjamin Baker for the Imperial Chinese Railways.

Sound engineering judgment and experience must be used in the interpretation and application of these specifications to particular cases, and the right is reserved to make modifications to them to suit specified conditions and variations and improvements in current practice.

Revised Specification for the Structural Portion of Heavy Cantilever Cranes.

Materials.

1. The whole of the structural steelwork, unless otherwise distinctly specified shall be of steel, conforming to the specification of the British Engineering Standards Committee. It shall be made by the Siemens-Martin open-hearth acid process, and have an ultimate tensile strength of from 28 to 32 tons per square inch, with an elongation of at least 20 per cent. in a length of 8 in.

Rivet steel shall have an ultimate tensile strength of from 26 to 30 tons per square inch, and a minimum elongation of 25 per cent. in a length of 8 diameters.

Steel for forgings shall have an ultimate tensile strength of from 28 to 32 tons per square inch, with a minimum elongation of 25 per cent. in a length of 8 diameters.

Cast steel shall be practically free from blowholes and other defects, and is to be annealed in all cases, and have an ultimate tensile strength of from 27 to 32 tons per square inch, with an elongation of at least 13 per cent. in a length of 6 in.

Dead Load.

2. The dead load shall be the whole of the structural steelwork, including all machinery, ropes and counterweight.

Working Loads.

3. The working loads shall be the maximum loads specified to be lifted at the different radii. To the moving loads the weights of the jenny, ropes and slings shall be added in computing the maximum stresses.

Test Loads.

4. The cranes shall be tested with loads, 20 per cent. in excess of the working loads at their specified radii, and shall be put through all the motions.

Wind Pressure.

5. The wind pressure shall be taken at 50 lb. per square foot when the crane is lifting no loads, and at 5 lb. per square foot when the crane is lifting the loads causing the maximum stresses. When the crane is lifting the test loads no wind pressure shall be assumed.

In all cases the wind pressure shall be assumed to act on a surface one and a half times the area of the surface seen in elevation, except when a surface shelters the portions behind it.

Momentum, Impact, Etc.

6. The various stresses caused by the hoisting of the different loads, the slewing and stopping of the loaded jib, and the racking of the loaded jenny, at their respective speeds, and the effect of brakes shall be considered in proportioning the sectional areas of different parts of the structure.

In no case shall these stresses be assumed to be less than the following additions to the direct static stresses :—

Lifting the Loads.—2½ per cent. of the load lifted, including slings and ropes, for all speeds up to 5 ft. per minute, and increased by 1 per cent. for each increase of 5 ft. per minute.

Slewing and Stopping the Loaded Jib.—1 per cent. of the total moving weight for speeds up to 20 ft. per minute at the pitch circle of the rack.

Racking.—2 per cent. of the moving weight, for speeds up to 20 ft. per minute.

Brakes.—2½ per cent. of the load being lifted.

Maximum Unit Stresses in Tension and Compression.

7. In no case shall the combined stresses under working conditions, wind or test loads, exceed three-tenths of the minimum ultimate strength of the material, nor shall they be more than the following:—

Under Dead Load, Working Loads, Impact and 5 lb. Wind.—The maximum stress shall not exceed $6\frac{1}{2}$ tons per square inch on the net section in tension or compression, but in no case shall the member in compression be subjected to a greater load than one-fifth of its ultimate strength when considered as a column.

Under Dead Load and 50 lb. Wind.—The maximum stress shall not exceed $7\frac{1}{2}$ tons per square inch on the net section in tension or compression, but in no case shall the member in compression be subjected to a greater stress than one-fourth of its ultimate strength when considered as a column.

Alternating Stresses.

8. Members subjected to alternate tension and compression shall be proportioned as a strut to resist the greater stress added to one-half of the lesser stress, except in the case of wind bracing, where the member shall be proportioned to resist the greater stress. The sum of the stresses shall be used in designing the connections.

Shearing, Bearing and Bending Stresses.

9. The shearing, bearing and bending stresses per square inch shall not exceed the following limits:—

(1) *In Truss or Lattice Girders, and Web or Flange Joints of Plate Girders*—

(*a*) For machine-driven rivets, or turned bolts and pins of a driving fit

Shearing stress ...	$\frac{3}{4}$ of the permissible tensile stress.		
Bearing stress ...	$1\frac{1}{2}$	do.	do.
Bending stress ..	$1\frac{1}{2}$	do.	do.

(*b*) For hand-driven rivets over 4 diameters in length. The number found by (*a*) shall be increased by 10 per cent.

(2) *In Plate Girders*—

Shearing stress in rivets ...	$\frac{7}{8}$ of the permissible tensile stress in the girder.			
Shearing stress in web plates ...	$\frac{1}{2}$	do.	do.	do.
Bearing stress on rivets ...	$1\frac{3}{4}$	do.	do.	do.

(3) *Bending Stress on Members Subject to Direct Tensile or Compressive Stresses.*—Where such stresses occur the member shall be proportioned to the algebraic sum of the stresses resulting from the direct stresses and three-fourths of the maximum bending stress, and the stress per square inch shall not exceed that permitted for the direct stresses. The member shall be considered as a beam freely supported at the ends, and the bending moment at the ends shall be assumed to be equal to that in the centre, but in the opposite direction.

Rollers.

10. The pressure in pounds per lineal inch of the live rollers of cast or rolled steel shall not exceed $300\,d$, where d is the mean diameter in inches.

Joints in Members.

11. All joints shall be fully covered and riveted to transmit the maximum stresses as a shearing stress through the rivets, except in the case of the tower legs which are machined, after each section is riveted, so as to bear throughout their whole faces, in which case they shall be covered and riveted to transmit at least one-third of the thrust as a shearing stress through the rivets. The cover plates shall have a sectional area 25 per cent. in excess of the sections joined.

Sectional Areas.

12. In estimating the sectional areas of the flanges of the jib, the rails and floor plating shall not be included in the area. For net sections the diameter of the rivet holes shall be taken as ⅛ in. larger than the nominal diameter of the rivet before driving for full-headed rivets, and ¼ in. for countersunk heads.

In plate girders, one-eighth of the gross section of the web plate shall be included in the sectional area of the flange, and the web covered to transmit horizontal stresses.

Foundations.

13. The pressure on the concrete of the foundations shall not exceed 10 tons per square foot under the working loads, dead load and wind pressure.

Constructional Details.

14. All members shall be designed of such form as to be accessible for inspection and painting, and to allow a free circulation of air through the member.

To allow for corrosive influences in the vicinity of manufacturing districts and near the sea, no angle, plate or bar shall be used of a less thickness than ⅜ in.

To minimise vibrations, and to provide a substantial and rigid frame, all members of the crane shall be formed of sections capable of resisting tension or compression, and the diagonal bracing of the tower shall be designed as tension members only.

The diameter of the roller-path shall be such that, under all conditions of loading, the centre of gravity of the loaded or unloaded jib shall not approach nearer the centre of the roller-path than one-tenth of the diameter.

All radial bars and roller frame of the live ring shall be formed of rigid members, with suitable tangential bars to maintain the relative motion of the parts of the frame.

To provide for emergency loading and a construction of a substantial and durable character, the centre pin, with the girders carrying it and their connections, shall be designed to resist a vertical and horizontal force of at least two-thirds of the maximum load lifted by the crane.

The size of the tower in plan shall be such that no tension shall exist in the foundations.

Workmanship.

15 The whole of the workmanship shall be of the highest class. The sheared edges of all plates and bars shall be planed, or machined, and all holes drilled. Hydraulic or power riveting shall be used where practicable. The abutting faces of the compression members of the tower shall be machined after the section is riveted and before fitting the cover plates, so as to ensure perfect contact on the abutting surfaces.

Painting.

16. (*a*) The whole of the structural steelwork before leaving the shop shall be scraped clean, and receive one coat of the best red-lead paint or boiled linseed oil.

(*b*) Where two surfaces are in contact, one of them shall receive one coat of paint, and all parts which are not accessible to painting after erection shall receive two coats of paint before being riveted together.

(*c*) After erection at site, the whole shall receive at least one finishing coat of the best oxide paint of an approved colour.

(*d*) All bright parts shall be coated with a suitable mixture of white lead and tallow.

General Specification for Structural Steelwork for Travelling Cranes.

Materials.

1. The whole of the structural steelwork, unless otherwise distinctly specified, shall be of steel, conforming to the specifications of the British Engineering Standards Committee. It shall be made by the Siemens-Martin open-hearth acid process, and have an ultimate tensile strength of from 28 to 32 tons per square inch, with an elongation of at least 20 per cent. in a length of 8 in.

Rivet steel shall have an ultimate tensile strength of from 26 to 30 tons per square inch, and a minimum elongation of 25 per cent. in a length of 8 diameters.

Cast steel shall be practically free from blow-holes and other defects, and is to be annealed in all cases, and have an ultimate tensile strength of from 27 to 32 tons per square inch, with an elongation of at least 13 per cent. in a length of 6 in.

Dead Load.

2. The dead load shall be the whole of the structural steelwork, including all fixed machinery, shafting, motors and operators' cage.

Working Loads.

3. The working loads shall be the maximum loads specified to be lifted. In computing the maximum stresses, the weights of the crab, ropes and slings shall be added to the moving loads.

Test Loads.

4. The cranes shall be tested with loads 50 per cent. in excess of the working loads, and shall be put through all their motions.

Permissible Working Stresses in Tension and Compression.

5. Under the various forces to which the crane may be subjected from lifting and lowering the full load, starting and stopping the longitudinal and cross travel of crane at the specified speeds, and the effect of brakes, the stresses shall not exceed the limits hereinafter specified.

(*a*) *Horizontal Chords of the Main Girders and Carriage Girders.*

The sectional areas shall be proportioned so that the maximum stress shall not exceed 6 tons per square inch on the net section, whether in tension or compression; nor shall the direct compressive stress per square inch on the gross section exceed the fraction $(0.95 - 0.003\,r)$ of this permissible stress or be greater than 85 per cent. of 6 tons. (r is the ratio of the length of the unbraced portion to its least radius of gyration.)

(*b*) *Diagonal Web Members of Lattice Main Girders.*

The sectional areas shall be proportioned so that the maximum stress shall not exceed the permissible stress on the net section for the diagonal next each end carriage, and $4\frac{1}{2}$ tons for the diagonal on each side of the centre of the span, whether they are in tension or compression, and, for diagonals lying between these points, the stress per square inch shall be reduced in a decreasing ratio from the higher limit at the end carriages to the lower limit at the centre of the girder. The working stress so determined shall be called

"the permissible tensile stress." For diagonals subjected to compressive stress the sectional areas shall be proportioned so that the stress per square inch on the gross section shall not exceed the fraction $(0.95 - 0.003\,r)$ of the permissible tensile stress, nor be greater than 85 per cent. of this stress. (r is the ratio of the length of the unbraced portion to its least radius of gyration.)

(*c*) *Shearing Stress on Web Plates.*

The shearing stress per square inch on the gross section shall not exceed one-half of the permissible stress; and the web plates shall be suitably stiffened against buckling at intervals not exceeding 5 ft. where the thickness of the web plate is less than one-sixtieth of the unsupported depth, and at all points of application of concentrated loads.

(*d*) *Rivets.*

The shearing stress per square inch on rivets in lattice girders shall not exceed three-quarters of the permissible tensile stress in the member, and the bearing stress one and one-half times the permissible tensile stress, and in plate girders seven-eighths and one and three-quarters respectively. The diameter of the rivet shall be the diameter before driving, and the bearing area shall be the diameter of the rivet multiplied by the minimum thickness of the bar or plate in the connection.

Alternating Stresses.

6. The sectional areas of members subjected to alternate tension and compression shall be proportioned as a strut for the greater stress in the member added to one-half of the lesser stress. The sum of the stresses shall be used in designing the connection.

Combined Bending and Direct Stress.

7. Members subjected to bending and direct stresses shall be proportioned so that the combined stress per square inch shall not exceed that allowed for the direct stress alone. The bending stress shall be calculated by assuming a freely supported span of three-quarters of a panel length, and the rail to distribute the load over a length not exceeding 12 in.

Joints in Members.

8. All joints shall be fully covered to transmit the whole stress as a shearing stress through the rivets. The covers shall have a sectional area of at least 25 per cent. in excess of the sections joined.

Sectional Areas.

9. In estimating the sectional areas of main girders, the rails and floor plating shall not be included in the estimated area; for net sections the diameter of the rivet holes shall be taken as $\frac{1}{8}$ in. larger than the nominal diameter of the rivet before driving for full headed rivets and $\frac{1}{4}$ in. for countersunk heads. The effective sectional area of tension members formed of angle, tee or channel bars, and connected by one leg shall be the net sectional area of the riveted leg added to one-half of the free leg; where they are connected by cleats, so as to be fully riveted in both legs, the whole sectional area may be taken after deducting rivet holes.

In plate girders, one-eighth of the gross section of the web plate shall be included in the sectional area of the flange, and the web covered to transmit horizontal stresses.

Camber.

10. The girders shall be constructed with a camber of not less than 1 in. in 60 ft. length.

Minimum Sections.

11 No section shall be used in the main and carriage girders of a less thickness than $\frac{5}{16}$ in., nor a smaller scantling than 3 in. by 3 in., and in the auxiliary girders and secondary bracing the thickness shall not be less than $\frac{1}{4}$ in., and the scantling smaller than $2\frac{1}{2}$ in. by $2\frac{1}{2}$ in.

Constructional Details.

12. Where the main or auxiliary girders are made of lattice construction all members shall be formed of rigid members capable of resisting tension or compression.

The main girders shall be rigidly secured to the end carriage girders by large gussets of ample dimensions and strength to keep the whole frame square and free from distortion.

The centres of wheels in the end carriages shall be spaced so that the crane will run true and not cross-bind, and shall, preferably, be spaced so that the centres are not less than one-fifth of the span. No member in compression shall have a greater unsupported length than 100 times its least radius of gyration or 45 times its least width.

Workmanship.

13. The whole of the workmanship shall be of a first-class character throughout, true to dimensions, and neatly finished.

All built members or girders shall be straight and out of wind, and when riveted the component parts shall fit closely.

All sheared edges of plates or bars shall be planed or machined, and the butting ends of compression members shall be planed or faced to bear throughout their whole faces.

The ends of all girders that butt or fit against other webs shall be finished true and square, so as to give a good bearing, and the end angles shall be flush with the ends of web plates.

All holes shall be drilled except where the connection is made by bolts, in which case the holes may be punched and reamed parallel or drilled after the parts are brought together and the bolts turned to be a driving fit.

Rivets must completely fill the holes and have large cup heads, and be machine driven wherever practicable. Countersinking shall be neatly done.

Painting.

14. (*a*) The whole of the structural steelwork before leaving the shop shall be scraped clean, and receive one coat of the best red-lead paint or boiled linseed oil.

(*b*) Where two surfaces are in contact, one of them shall receive one coat of paint, and all parts which are not accessible to painting after erection shall receive two coats of paint before being riveted together.

(*c*) After erection at site, the whole shall receive at least one finishing coat of the best oxide paint of an approved colour.

(*d*) All bright parts shall be coated with a suitable mixture of white lead and tallow.

General Specifications for Bridge Superstructures.

CONTENTS.

Materials for Bridges.

General

1. The whole of the structural bridgework, except bedplates, machinery and turning gear for swing bridges shall be of rolled steel.

The bedplates, machinery and turning gear shall generally be of cast steel or iron, or forged steel.

Parapets and ornamental work may be of cast iron.

All steel shall comply with the Specifications of the British Engineering Standards Committee for Structural Steel.

Rolled Steel.

2. All steel shall be made by the open-hearth acid process by approved manufacturers. It shall be free from laminations and surface defects.

Strips cut lengthwise or crosswise shall have an ultimate tensile strength of not less than 28 tons and not more than 32 tons per square inch of original section, with an elongation of not less than 20 per cent. in a length of 8 in., and when heated uniformly to a blood red and cooled in water of 80 deg. Fahr., strips $1\frac{1}{2}$ in. wide must stand bending double in a press to a curve of which the inner radius is $1\frac{1}{2}$ times the thickness of the steel tested.

Steel for forgings shall have an ultimate tensile strength of from 28 to 32 tons per square inch, with an elongation of at least 25 per cent. in 8 diameters.

Rivet Steel

3. Rivet steel shall have an ultimate tensile strength of not less than 26 tons and not more than 30 tons per square inch of original section, with an elongation of not less than 25 per cent. in a length of eight times the diameter.

Cast Steel.

4. Steel castings shall be made of open-hearth steel without injurious defects, and practically free from blow-holes. All castings shall be annealed, and shall have an ultimate tensile strength of not less than 27 tons nor more than 32 tons per square inch of original section, with an elongation of not less than 16 per cent. in a length of 2 in. and 13 per cent. in 6 in.

Cast Iron

5. All castings shall be of the best tough gray metal of such a strength that a bar 1 in. thick by 2 in. deep placed upon bearings 3 ft. apart will sustain without fracture a weight of 27 cwt. placed at the centre with a deflection of not less than $\frac{1}{3}$ in.

Wrought Iron.

6. (*a*) When iron is used it shall be of a fibrous quality, free from all defects, and shall stand such forge tests as shall prove the quality of the material used and its fitness for the service.

(*b*) Bars over 2 in. in diameter and angle and other shaped iron shall have an ultimate tensile strength of not less than 22 tons per square inch of original section, with an elongation of at least 12 per cent. in a length of 8 in., and be capable of bending double when cold without cracking to a curve of which the inner radius is twice the thickness of the piece tested.

(*c*) Plate iron shall have an ultimate tensile strength of not less than 21 tons per square inch of original section, and an elongation in a length of 8 in. of at least 8 per cent. with the grain, and 17 tons per square inch and 3 per

cent. elongation across the grain. The cold bend shall be 35 deg. with the grain and 15 deg. across for ½-in. plate, and proportionate amounts for other thicknesses, the radius of the curve being equal to the thickness of the plate.

(*d*) The wrought iron used for rivets, bolts and bars under 2 in. in diameter shall have an ultimate tensile strength of not less than 23 tons per square inch of original section, with an elongation of not less than 20 per cent. in 8 in., and shall bend double when cold without cracking.

Timber.

7. All timber shall be of the best quality, sawn true and out of wind, full size and free from shakes, large or loose nuts, decayed wood, sap, worm holes, or any other defect which would impair its strength or durability.

Painting.

8. (*a*) The whole of the structural steelwork before leaving the shop shall be scraped clean, and receive one coat of the best red-lead paint or boiled linseed oil.

(*b*) Where two surfaces are in contact, one of them shall receive one coat of paint, and all parts which are not accessible to painting after erection shall receive two coats of paint before being riveted together.

(*c*) After erection at site, the whole shall receive at least one finishing coat of best oxide paint of an approved colour.

(*d*) All bright parts shall be coated with a suitable mixture of white lead and tallow.

Railway Bridges.

Loading.

Dead-Load.

9. The dead load must not be less than the actual weight, and shall consist of the whole weight of the steel superstructure, permanent way and ballast, if any. The weight of the ordinary permanent way, including 95 lb. rails, chairs, cross sleepers, etc., shall be taken at 1½ cwt. per lineal foot of single line; and for an open floor the weight of the wooden cross sleepers, permanent way and guard rails, shall be taken at 4 cwt. per lineal foot of single line. Where ballast is used with ordinary permanent way, the average depth shall be taken at 12 in. and assumed to weigh 120 lb. per square foot of floor.

For spans of less than 200 ft. the total dead load shall be assumed to act at the loaded chord. For spans of 200 ft. and over, the total dead load will be distributed at top and bottom chords as follows:—

(1st.) *On Loaded Chords:*

(*a*) One-half load resulting from weight of trusses.

(*b*) Weight of horizontal wind bracing in plane of chords.

(*c*) Weight of floor system, permanent way, etc.

(*d*) One-half of load resulting from weight of cross bracing in the case of a deck bridge.

(2nd.) *On Unloaded Chords:*

(*a*) One-half load resulting from weight of trusses.

(*b*) Weight of horizontal wind bracing in plane of chords.

(*c*) One-half of load resulting from weight of cross bracing in the case of a deck, or the whole of it in the case of a through bridge.

Live Load.

10. The structure shall be designed to carry a moving load for each track consisting of two engines coupled, at the head of a uniformly distributed train load.

The live-load stresses will be the maximum stresses produced by the rolling load considered as stationary or as moving in either direction. In double-track structures one track or both will be considered loaded, whichever may produce the greater stresses, and the trains will be supposed to move either in the same or in opposite directions.

The diagram of train loads shall be furnished with the inquiry and be shown upon the stress diagrams.

Where no train load is supplied with the inquiry, the live load per track of standard 4 ft. 8½ in. gauge shall be computed as follows:—

For Main Girders:

Distributed rolling load per track = 1½ tons per foot run + an excess rolling load to occupy any position on the bridge at the same time equal to $\left(15 \text{ tons} + \dfrac{\text{span in feet} - 10}{10}\right)$, but not greater than 25 tons.

For Cross Girders:

The load on each cross girder from each track—

For all cross girders up to 7 ft. centres = 19 tons.

For all cross girders beyond 7 ft. centres = 1½ tons per foot run of track plus an excess load of 10 tons $-\left(\dfrac{10 - \text{centres of cross girders in feet}}{5}\right)$ but not greater than 10 tons.

Wind Pressure.

11. The wind shall be assumed acting in either direction, horizontally, and to be blowing at a slight angle to the axis of the bridge so as to take effect on the exposed areas of the floor, and of both windward and leeward girders, except when the latter is temporarily screened by a passing train. One-half of the exposed surface of the leeward girder shall be included in the total area acted upon by the full wind pressure, except when the distance between the main girders is more than twice their depth, when the whole exposed area of leeward girder shall be taken.

With no train on the bridge the wind pressure shall be assumed to be 56 lb. per square foot, and with a train on the bridge at 30 lb. per square foot of exposed surfaces of train and bridge. The train shall be taken at 10 square feet per lineal foot, and the centre of pressure of train surface at 7½ ft. above rail level. The wind pressure on the train shall be treated as a moving load. The maximum stresses resulting from either condition to be taken in determining the necessary sectional areas of the parts.

In providing the necessary anchorage for the structure, the bridge shall be assumed to be covered with a train of empty passenger carriages weighing 10 cwt. per lineal foot; and in the case of a double-line bridge, the leeward track only to be loaded.

Momentum of Train.

12. Special attention shall be given to the details of the structure to provide for the longitudinal stresses resulting from the tractive force of the engines or from the sudden application of continuous brakes to the train while on the bridge, and the horizontal force resulting from such action shall be taken as one-fifth of the weight of the train.

Centrifugal Force.

13. When a bridge is on a curve, the resulting horizontal stresses due to the centrifugal action of the rolling load shall be provided for.

The centrifugal force for each degree of curvature shall be assumed to be 1 per cent. of the maximum rolling load on all tracks for a speed of 30 miles per hour and under, and 1 per cent. shall be added for each increase in speed of 10 miles per hour.

The centrifugal force shall be assumed to act at 5 ft. above the level of the rails.

The radius in feet R shall be reduced to degrees in curvature D by the following formula: $D = \frac{5730}{R}$.

In summating the resulting stresses, the wind shall be assumed to be acting in the same direction as the centrifugal force.

Working Stresses.

Impact.

14. The following working stresses have been proportioned to allow for dynamic action of the live load on lightly-loaded girders or members of girders :—

Permissible Maximum Stresses.

15. All bridgework and trestle piers shall comply with the whole of the following conditions :—

(1st.) The combined stresses, resulting from the rolling load, dead load, wind, momentum and centrifugal forces, shall not produce a greater tensile stress than one-half of the elastic limit, or equal to three-tenths of the minimum ultimate tensile strength of the material, nor more than the corresponding compressive, shearing, bearing and bending stresses as hereinafter set forth ; but

(2nd.) The combined stresses, resulting from the rolling load and dead load alone, exclusive of wind, momentum and centrifugal force, shall not produce greater tensile stresses than those tabulated below.

Tensile Stresses

16. *For Main Girders, Cross Girders and Rail Bearers of Plate Construction.*

Under 20 ft. span	$4\frac{1}{2}$ tons per square inch
20 ft. and under 25 ft. span	$4\frac{3}{4}$ do.
25 ft. and under 30 ft. span	5 do.
30 ft. and under 50 ft. span	$5\frac{1}{4}$ do.
50 ft. and under 80 ft. span	$5\frac{1}{2}$ do.

For Truss and Lattice Girders.

80 ft. and under 160 ft. span :	
Bottom chords	$5\frac{1}{2}$ tons per square inch
Diagonals	$4\frac{1}{2}$ to $5\frac{1}{2}$ do.
160 ft. and under 200 ft. span :	
Bottom chords	$5\frac{3}{4}$ do.
Diagonals	$4\frac{1}{2}$ to $5\frac{3}{4}$ do

200 ft. to 400 ft. span :

Bottom chords	... 6 to 7	tons per square inch
Diagonals	... $4\frac{1}{2}$ to 7	do.

All spans :

For wind-bracing	$8\frac{1}{2}$	do.
For floor suspenders	$2\frac{1}{2}$	do.

NOTE.—The $4\frac{1}{2}$ tons stress on the diagonals will apply to those at the centre portion of the span and to the counterbracing at the same point. The higher stresses will apply to those at the end portions of the span, where the variations of stress are not so great. Intermediate diagonals will be subject to stresses lying between the two limits.

Compressive Stresses.

17. For plate girders, the gross area of the compression flange shall not be less than that of the tension flange, nor shall the compressive stresses per square inch be more than 85 per cent. of the corresponding specified tensile stress.

For truss or lattice girders, the compressive stress per square inch shall in the case of riveted members not exceed the fraction $(0.95 - 0.003\ r)$ of the corresponding specified tensile stress, nor in the case of pin-connected members the fraction $(0.95 - 0.0045\ r)$, where r is the ratio of the length of the unbraced portion of a member to its least radius of gyration; nor in any case shall it exceed 85 per cent. of the said tensile stress. No compression member shall have a greater length than 100 times its least radius of gyration or 45 times its least width, except for wind-bracing, which may have a length not exceeding 120 times its least radius of gyration.

Alternating Stresses.

18. Members subject to alternate tension and compression shall be proportioned as struts, to resist the greater stress added to one-half of the lesser stress, except in the case of wind-bracing, where the member shall be proportioned to resist the greater stress. The sum of the stresses shall be used in designing the connections.

Shearing, Bearing and Bending Stresses.

19. The shearing, bearing and bending stresses per square inch shall not exceed the following limits:—

(1.) *In Truss or Lattice Girders, and all Web or Flange Joints in Plate Girders.*

(*a*) For machine-driven rivets and turned bolts or pins of a driving fit.

Shearing stress ...	$\frac{3}{4}$	of the permissible maximum tensile stress per square inch in the girder or member.
Bearing stress ...	$1\frac{1}{2}$	ditto
Bending stress ...	$1\frac{1}{2}$	ditto

(*b*) For hand-driven rivets having a length over four diameters.
The number found for (*a*) to be increased by 10 per cent.

(*c*) For ordinary black bolts.
The number found for (*a*) to be increased by 25 per cent.

(2.) *In Plate Girders.*

Shearing stress in rivets ...	$\frac{7}{8}$	of the permissible maximum tensile stress per square inch in the girder.
Shearing stress in web plate ...	$\frac{1}{2}$	ditto
Bearing stress on rivets ...	$1\frac{3}{4}$	ditto

(3.) *Bending Stress on Members Subject to Direct Tensile or Compressive Stresses.* Where such stresses occur the member shall be proportioned to the algebraic sum of the stresses resulting from the direct stresses and three-fourths of the maximum bending stress, and the stress per square inch shall not exceed that permitted for the direct stresses. The member shall be considered as a beam freely supported at the ends, and the bending moment at the ends shall be considered equal to that in the centre but in opposite direction.

Rollers and Bedplates.

20. The pressure in pounds per lineal inch on rollers of rolled steel shall not exceed 300 d, where d equals the diameter of the rollers in inches; in the case of live rollers under swing bridges the pressure shall not exceed three-fourths of this amount.

Bedplates and rockers shall be of sufficient area and strength to distribute the load over the masonry without exceeding a pressure of 16 tons per square foot for hard stone, of 20 tons per square foot for granite, and 10 tons per square foot for cement concrete (4 – 2 – 1).

Wrought Iron-work.

21. Where wrought iron is used for any girder or member of a bridge, the working stresses shall be 80 per cent. of those specified in the case of steel for members subject to tensile and bending stresses, and for short compression members; 85 per cent. for long compression members, and 90 per cent. for members subject to shearing and bearing stresses.

Wind and Centrifugal Force.

22. Where the stresses resulting from the dead and live loads are combined with those due to the wind alone, or with the wind and centrifugal force, the preceding working stresses per square inch may be increased 25 per cent., but in no case shall they exceed three-tenths of the minimum ultimate tensile strength of the material.

Connections.

23. Connections shall be proportioned to develop the full strength of the member, notwithstanding that the calculated stress may be less.

NOTE.—Where bridges or trestle piers have to be constructed to the requirements of the Board of Trade, the combined maximum stresses shall not exceed $6\frac{1}{2}$ tons per square inch in tension or compression in the case of steel bridges, and 5 tons in the case of wrought-iron bridges; and the wind pressure shall be taken at 56 lb. per square foot of exposed surface on the loaded bridge and trestle.

STRUCTURAL DETAILS.

Types of Bridges.

24. For spans of 16 ft. and under, rolled beams may be used; for spans from 16 ft. to 80 ft., plate girders shall be used; for spans from 80 ft. to 200 ft., riveted truss or lattice girders shall be used; and above 200 ft., either riveted or pin-connected truss or lattice girders may be used.

Minimum Sections.

25. No shape weighing less than 6 lb. per lineal foot shall be used, nor any plate or bar less than $\frac{5}{16}$ in. in thickness when both sides are accessible for painting, nor less than $\frac{3}{8}$ in. when only one side is accessible for painting. The web plate of plate girders shall not be less than $\frac{3}{8}$ in. in thickness. The unsupported width of any plate subjected to compression shall not exceed thirty times its thickness, except in the case of flange plates of trough-shaped booms and posts, where it may be forty times its thickness. No angle less than 3 in. by $2\frac{1}{2}$ in. shall be used in the main members of girders or trusses, or in any

member having rivets $\frac{7}{8}$ in. in diameter. No angle less than $2\frac{1}{2}$ in. by $2\frac{1}{2}$ in. shall be used in any part of a bridge structure.

End angles connecting rail bearers or cross-girders shall not be less in thickness than the thickness of the web plates.

Bedplates shall not be less than $\frac{3}{4}$ in. in thickness.

No eye bars less than $\frac{3}{4}$ in. nor over 2 in. in thickness shall be used. The minimum section shall be 4 in. by $\frac{3}{4}$ in. The depth of eye bars for chords and main diagonals shall not be less than $\frac{1}{35}$ of the length of the horizontal projection of the distance between the points of support.

No main pins shall be less than $3\frac{1}{2}$ in. in diameter, nor less than three-quarters of the width of the widest bar attached to them.

Rivets. 26. The pitch of rivets in the direction of the stress shall not exceed 8 in. in any case, nor be more than sixteen times the thickness of the thinnest outside plate or angle bar, nor be less than 3 diameters, and not more than forty times the thickness of the thinnest outside plate at right angles to the stress. The distance from the centre of the rivet or bolt hole to the edge of a plate or bar shall not be less than $1\frac{1}{2}$ diameters in the case of machined or rolled edges, nor $1\frac{3}{4}$ diameters in the case of sheared edges, nor exceed eight times the thickness of the plate.

At the ends of plate girder flange plates the pitch of rivets shall not exceed $4\frac{1}{2}$ diameters for a length sufficient to provide a number of rivets whose combined sectional areas shall be equal to the net sectional area of the flange plate. The flange plate shall be of such length that one-half of these rivets shall be beyond the theoretical end of the plate. Where webs are built up of two or more plates, the rivets, which are used solely for making the several thicknesses act as one plate, shall not be spaced more than 12 in. apart. Such compound web plates shall not be used where the total thickness is less than 1 in.

At the ends of riveted columns or struts for a length equal to twice the width of the member, the pitch of rivets shall not exceed $4\frac{1}{2}$ diameters.

Provision for Temperature. 27. Freedom for expansion and contraction due to change of temperature shall be provided in all spans at the rate of 1 in. for every 100 ft. in length.

Bridges shall be provided with suitable bearing plates riveted to the flanges and bolted through the bedplates to the masonry at one end, and be free to move longitudinally at the other end. Suitable expansion bearings shall be provided at one end of each span.

The expansion bearings shall be so designed as to permit of inspection and lubrication. They shall permit of a free movement in a longitudinal direction sufficient to take up the extreme variations in length due to temperature changes and deflection, and at the same time to prevent any transverse motion or uplifting of the end of the span.

Camber. 28. Bridges of 100 ft. span and upwards shall be constructed with a camber of 1 in. for every 100 ft. in length. With parallel chords sufficient camber will be obtained by making the top chord sections longer than the corresponding bottom chord sections by $\frac{1}{8}$ in. for every 10 ft. of length.

Plate girders shall not be given any camber.

Joints. 29. The butting ends of all spliced members, whether in tension or compression, shall bear evenly throughout their whole faces and be fully covered and riveted to transmit

the whole stress through the splice. Web splices shall have double covers of sufficient width to admit of sufficient rivets to transmit the whole of the shearing stress at the joint.

The sectional area of covers shall be 25 per cent. in excess of the sections spliced.

Handrails. 30. Where handrails are required on bridges, they shall be constructed at least $4\frac{1}{2}$ ft. high above rail level of two lines of gas pipes of 1 in. internal diameter, with suitable standards not more than 5 ft. apart, or of open lattice work or plates $\frac{1}{4}$ in. in thickness, suitably stiffened.

Trough Flooring. 31. Where trough flooring is used for bridge floors, it shall be designed on the assumption that the wheel loads are distributed over 5 ft. run of the floor if the sleepers are parallel with the flutes of the troughs, and 10 ft. if they are at right angles to the flutes, without the fibre stress exceeding that specified for the span. A suitable stiffening girder shall be used in the centre of the span where the troughs carry a double track.

Wind-Bracing and Cross-Bracing. 32. Wind-bracing and cross-bracing between main girders or struts shall be formed of rigid members capable of resisting tension or compression.

General Data Assumed for Calculations.

Effective Spans and Depths. 33. For the purposes of calculating the moments, stresses, shears and working strengths, the effective lengths and depths shall be taken as follows:—

For Main Girders.—The centres of bearing plates in the case of riveted plate or truss girders, and the centres of end pins in the case of pin-connected trusses.

For Cross Girders.—The centres of the main longitudinal girders or trusses.

For Rail Bearers.—The centres of cross girders.

For Struts.—The centres of the vertical web plates of booms with riveted connections and the centres of pins with pin connections, where the web consists of a single system, but where the web consists of more than one system the length shall be taken between the points of intersection or between the points of intersection and the centre of vertical web plates or pin.

For Compression Flange and End Posts.—Between the points of intersection of the vertical or horizontal bracing with the flanges in weakest plane of bending.

Bending in Pins.—Between centres of bearings.

Effective Depths.—The effective depths of riveted plate or truss girders shall be taken as the centre of gravity of the upper and lower flanges, and, in the case of pin-connected trusses, the centres of pins, but not in any case to be more than the distance over the angles connecting the vertical web plates with the horizontal flange plates. The depth between the centres of the horizontal rows of rivets shall be used in calculating the horizontal shearing forces on the rivets connecting the flange angles to the web plates in plate girders.

Sectional Areas. 34. The net sectional area shall be taken for all tension members, and shall be determined by a plane cutting the member square across at any point. The greatest number of holes which can be cut by any such plane, or whose centres come nearer than $1\frac{1}{2}$ in. to it, are to be deducted from the gross section when computing the net area.

The gross sectional area shall be taken for all compression members.

The bearing and shearing areas of rivets shall be calculated on the diameter before driving.

The shearing stress on the web plates of plate girders shall be calculated on the gross sectional area of the full depth of the plate.

In plate girders the flanges shall be calculated as resisting the whole of the bending stresses and web plates the whole of the shearing stresses, but one-eighth of the web plates may be included in the estimated sectional areas of the flanges, if the web plates are suitably covered to transmit horizontal stresses.

In deducting for rivet or bolt holes, the diameter of the hole shall be taken as $\frac{1}{8}$ in. greater than the nominal diameter of the rivet or bolt for full-headed rivets or bolts and $\frac{1}{4}$ in. larger for countersunk holes.

Loads.

35. All dead loads shall be assumed to be evenly distributed in the case of plate girders, except the load brought on the main longitudinal girders from the cross girders when they exceed 10 ft. centres. In the case of lattice or truss girders the dead load shall be assumed to be collected at the panel joints.

Live loads shall, in the case of lattice or truss girders, be considered to cover the panel in advance of the panel joint being considered, but the half load will be ignored in the calculations.

Riveting in Webs.

36. In calculating shearing and bearing stresses on web rivets of plate girders, the whole of the shear acting on the side of the panel next the abutment shall be considered as being transferred into the flange angles in a distance equal to the effective depth of the girder.

Provision shall be made for local shear from heavy wheel loads, and the rivet spacing in top flanges of deck plate girders and rail bearers shall not exceed 4 in.

Vertical Cross Bracing Between Struts.

37. The vertical cross bracing between struts shall be proportioned to carry 50 per cent. of the panel load due to wind, and the struts shall be calculated to resist any bending stresses from the wind loads.

Rolled I Beam Spans.

Depth.

38. Rolled joists used as longitudinal girders shall, preferably, have a depth of not less than one-twelfth of the span.

Construction with Open Floor.

39. Rolled joist spans may be constructed with either one or two joists per rail.

With single joists per rail the spacing should be $6\frac{1}{2}$ ft. centres, with rigid cross struts and diagonal bracing between the top flanges, unless the span is under 10 ft., when the diagonal bracing may be omitted.

With two joists per rail the spacing should not exceed $2\frac{1}{2}$-ft. centres for the two joists under the rail. No diagonal bracing is required, but the rigid cross struts must extend across the four joists under each track at the ends of the spans and also at mid-span when the length of span exceeds 10 ft.

Construction with Plated Floor.

40. Rolled joist spans with plated floor riveted to the top flanges may be constructed with either one or two joists per rail spaced at the same centres as with the open floor.

General.

41. Each joist shall have at each end a pair of end stiffening angles riveted to the web and fitted tightly between the top and bottom flanges.

Suitable bearing plates to distribute the load on the abutments shall be provided at each end of the joists.

Where handrails are required they shall be carried on suitable longitudinal rolled joists, and shall be either of gas tubes with standards or of plate construction as required.

PLATE GIRDER SPANS.

Depth.

42. Plate girders shall, preferably, have a depth of from one-tenth to one-twelfth of the span.

Splices.

43. All plate girders, whenever it is practicable, shall be built without splices; but, where this is unavoidable, the smallest number of splices shall be adopted.

Flanges.

44. Whenever practicable, at least one-half of the flange section shall be contained in the angles, or else the heaviest section of angles shall be used, and the number of flange plates reduced to a minimum. To obtain an even distribution of stress over the cross section of the flange plates, they shall not project more than 8 in. or sixteen times their thickness beyond the outer line of rivets through the flange angles.

The compression flange shall be stiffened laterally by cross-bracing frames in the case of deck spans, and triangular brackets extending from the top flange to each cross girder in through spans, at intervals of not more than fifteen times its width. The length of the compression flange shall not exceed forty times its width.

Main girders of plate construction shall, preferably, have one flange plate extending from end to end in the compression flange.

Web Plates and Stiffeners.

45. Web plates shall have angle-bar stiffeners riveted on both sides at the ends and inner edges of the bearing plates, and at all points of local and concentrated loads, and also at points throughout the length of the girder, generally not farther apart than the depth of the girder, with a maximum spacing of 6 ft., when the thickness of the web is less than one-sixtieth of the unsupported distance between the flange angles.

All stiffeners shall bear tightly at top and bottom against the flange angles. All stiffeners over the bearing plates shall have packings under them of the same thickness as the flange angles and as wide as the stiffener angles, but intermediate stiffeners shall, preferably, be joggled over the flange angles unless the latter exceed $\frac{5}{8}$-in. in thickness. Where practicable, stiffeners shall be placed at web joints. The stiffeners and the rivets connecting them to the web plate should be of sufficient area to take two-thirds of the vertical shear at the point of attachment of stiffeners to web plate.

Stiffening angles over the bearing plates shall in no case be less than $3\frac{1}{2}$ in. by $3\frac{1}{2}$ in. by $\frac{3}{8}$ in., but must have sufficient area to carry the entire shear without exceeding the specified intensity of working stress, no reliance being placed on the packings. They shall be proportioned as struts having a length equal to three-fourths of the depth of the girder.

Girders will be neatly finished at the ends. They will, generally, have a plate, corresponding in width to the flange plates, riveted to the end angles.

Cross-Bracing and Deck Spans.

46. Cross-bracing, consisting of complete frames, shall be used at the ends and at intermediate points where needed for wind and centrifugal force.

Lateral Bracing.

47. In spans with open floors, horizontal diagonal bracing shall extend from end to end of sufficient section to resist the wind and centrifugal force. This bracing shall, preferably, be of rigid members. In plated floors this diagonal bracing may be omitted.

Riveted Truss or Lattice Girders.—Through Bridges.

General Design. 48. The main girders of through bridges shall, preferably, be of the single intersection type, with inclined end posts and ties and vertical struts and suspenders.

Cross girders shall be riveted to the vertical struts and suspenders, and a cross girder shall be secured to ends of main girders to support the rail bearers.

Rail bearers shall be placed under each rail and riveted to the cross girder webs. The deck shall be covered with buckled plates, riveted to the floor girders and to suitable intermediate supports. The buckled plates shall not be less than $\frac{5}{16}$ in. in thickness, with a buckle of at least $2\frac{1}{2}$ in., and preferably placed with the buckle downwards. Suitable provision shall be made for draining the floor of the accumulation of rain water.

Horizontal diagonal bracing shall be fixed between the top booms of main girders of the necessary strength to transmit the wind pressure safely to the portal bracing between the end posts, and of sufficient rigidity and stiffness to keep the booms in line. Where there is an open floor, rigid horizontal diagonal bracing shall be fixed between the bottom booms of main girders to transmit the lateral stresses to the piers or abutments. The lower diagonal bracing shall be rigidly secured to the rail bearers, so as to transmit the longitudinal thrust due to train momentum through the diagonals to the main girders and to relieve the cross girders of horizontal bending.

Cross-bracing of the maximum depth permissible with the required headroom shall be fixed between the tops of struts, with knee brackets riveted at top corners to struts and cross-bracing and at bottom corners to struts and cross girders, so that a rigid frame is formed at each strut or vertical suspender.

Portal bracing of the maximum depth permissible with the required headroom shall be riveted to the end posts. Rigid knee brackets shall be riveted to the portal bracing and end posts. In determining the sectional area of the end posts, provision must be made for the bending stresses due to the wind pressure. The end posts shall be considered as fixed at the ends, and the leverage from bottom of end post to underside of portal bracket reduced by one-half.

General Proportions. 49. The depth of main and cross girders shall not be less than one-tenth of the span and shall, preferably, be one-eighth. Rail bearers shall have a depth of not less than one-twelfth of their span.

The centres of the main girders shall not be less than one-twentieth of the span, and the height of main girders not more than three times the width between their centres.

Construction of Main Girders. 50. The booms and end posts shall, preferably, be of trough section. The top booms and end posts shall have edge angles riveted to the edge of the vertical plates of the troughs. Suitable plate diaphragms shall be riveted between the vertical side plates of booms. The width of booms shall not be less than one-twelfth of the unsupported distance. Provision shall be made for draining the trough booms of the accumulation of rain water.

Struts shall, generally, be of four angles, with or without side plates and a web plate; but where lacing bars are substituted for the web plate, they shall conform to the rules of lacing bars for compression members.

Ties shall, as far as practicable, be constructed of rigid members, but they may be of rolled flat bars, except near the centre, where they must be formed of rigid

members. Counter-bracing shall be of similar construction to the centre ties. Distance pieces shall be used between the plates forming long ties to reduce vibration.

The open side of long compression members shall be stayed with intermediate tie plates or bracing where necessary. The tie plates shall have a thickness of not less than one-fiftieth of their unsupported width, except where they are stiffened with angle bars, when they may be $\frac{5}{16}$ in. The length of tie plates at the ends of laced struts or lateral bracing shall not be less than the vertical side plates of the main booms. Single lacing bars shall, preferably, have a thickness of not less than one-fiftieth of the distance between the centres of the rivets connecting them to the main angles and double lacing bars one-sixtieth. The distance between connections of lacing bars shall not exceed eight times the least width of the segments connected.

Vertical suspenders shall be composed of rigid members, and shall be proportioned to take three-quarters of the stress as a compressive stress.

All sections shall, as far as possible, be symmetrical about the centre line of stress, and all rivets grouped symmetrically about the same line.

Where angle bars connected by one blade are used as ties, the sectional areas shall be taken as follows:—For equal sides angle bars, 75 per cent. of net sectional area; for angle bars with sides in the proportion of 2 to 1 and connected by longer side, 90 per cent.; intermediate sizes shall be interpolated.

Construction of Floor Girders.

51. Cross girders and rail bearers shall, preferably, be composed of four angles and a web plate without flange plates and the details shall, generally, conform to the rules for plate girders.

Half Through Bridges.

General.

52. The details shall, generally, conform to the requirements for through bridges, but all struts shall be formed with plate webs and knee brackets of the largest dimensions permissible with the required clearances shall be riveted to each strut and cross girder.

Deck Bridges.

General.

53. The details shall, generally, conform to the requirements for through bridges. Rigid cross frames and diagonal bracing shall be provided as for deck plate girders.

Workmanship.

General.

54. The whole of the workmanship shall be of a first-class character throughout and true to dimensions.

All built members or girders shall be straight and out of wind and when riveted the component parts shall fit closely.

All girders shall be neatly finished wherever exposed to view.

Planing, Machining, and Fitting of Sheared Edges.

55. All sheared edges of plates or bars shall be planed or machined.

The butting ends of compression members shall be planed or faced to bear throughout their whole faces.

The ends of all girders that butt or fit against other webs shall be finished true and square or to exact level required, so as to give a good bearing and end angles shall be flush with ends of web plates.

All packings and cover plates must fit sufficiently close to the flanges at their ends to be sealed against the admission of water when painted.

All web stiffeners shall be fitted to bear tightly against the flange angles.

Punching, Drilling and Riveting. 56. All rivet, bolt and pin holes shall be drilled.

All rivet holes shall be $\frac{1}{16}$ in. larger in diameter than the nominal size of the rivet.

Rivets must completely fill the holes and have large cup heads, and be machine driven wherever practicable. Countersinking shall be neatly done.

Eyebars. 57. Eyebars shall be formed without welding and shall be slightly stronger in the head than in the body of the bar.

The heads shall be made by upsetting, rolling or forging into shape. A variation from the specified dimensions of the heads will be allowed, in thickness of $\frac{1}{32}$ in. below and $\frac{1}{16}$ in. above that specified and in diameter $\frac{1}{4}$ in. in either direction.

Eyebars must be perfectly straight before boring. All eyebars shall be annealed.

Loop Ends to Bars. 58. Where unavoidable, welding will be allowed to form the loop ends of minor bracing bars.

Screw Ends to Bars. 59. All screw ends to bars shall be at least $\frac{1}{16}$ of the diameter larger at the base of the thread than in the body of the bar and the enlarged ends shall be formed without welding. This increased diameter shall be taken in the case of bars used without enlarged ends.

Riveted Tension Bars. 60. Riveted tension bars with pin connections shall have a net area through pin-hole of not less than one and a-half times the net area in the body of the bar and between the pin-hole and the end of the bar of at least four-fifths of the net area. Sufficient rivets shall be provided to make the thickening plates at the pin-hole effective.

The length from edge of pin-hole to the end of tension bar shall not be less than the diameter of the pin.

Pins. 61. All pins shall be turned straight and smooth to a gauge and shall fit the pin-holes to $\frac{1}{50}$ in. They shall be turned to a smaller diameter at the ends for the thread and driven to place with a pilot nut where necessary to preserve the threads.

Rollers and Bedplates. 62. Rollers shall be turned accurately to gauge and be finished perfectly round and to the correct diameter from end to end. The tongues and grooves in the plates and rollers must fit closely to prevent lateral motion.

Roller beds and expansion bearings shall be planed.

Steel Trestles or Piers.

Loading.

Dead Load. 63. The dead load shall be calculated as specified for the main girders of the bridge, with the addition of the weight of the pier.

Live Load. 64. The live load shall be calculated as specified for the main girders of the bridge.

Wind Pressure. 65. The wind pressure shall be calculated as specified for the main girders of the bridge, with the addition of the pressure on the exposed surfaces of the pier, the assumed wind pressure being 30 lb. per square foot with the train on the bridge and 56 lb. per square foot without the train. No member of the pier on one side shall be assumed to shelter the corresponding member on the opposite side.

Momentum of Train and Centrifugal Force.

66. These forces shall be calculated as specified for the main girders of the bridge.

Temperature.

67. Where the ends of the main girders of the bridge rest upon sliding bedplates during movements due to temperature, the sliding friction shall be assumed to be 25 per cent. of the dead load upon the bedplates, and shall be added to the longitudinal force due to the application of brakes to a train upon the bridge.

Working Stresses.

Impact.

68. The effects of impact of the live load is provided in the reduced working stresses given in the following paragraphs.

Permissible Maximum Stresses.

69. (*a*) *Under dead and live loads exclusive of wind.*

The compressive stresses per square inch shall be a fraction of the permissible tensile stress per square inch in the chords of the longitudinal main girders supported by the pier, and shall be reduced by the same formula as specified for the compression members of the main girders.

(*b*) *Under combined dead and live loads and 30 lb. wind and centrifugal force; or, combined dead load and 56 lb. wind; or, combined dead and live loads, momentum and temperature.*

Legs. The permissible compressive stresses per square inch shall be increased by 25 per cent. over those allowed for the dead and live loads alone.

Diagonal transverse or longitudinal bracing:

—	In tension.	$8\frac{1}{2}$ tons per square inch.
—	In compression.	Reduced by the formula for compression members, using $8\frac{1}{2}$ tons as the specified tensile stress.
Anchorage Bolts.	In tension.	Three-fourths of the permissible tensile stress per square inch in the chords of the main girders.

Shearing, Bearing and Bending Stresses.

70. The shearing, bearing and bending stresses per square inch shall be calculated as specified for the lattice main girders of the bridge.

General Design.

Construction

71. The tower or pier shall be formed of four legs or columns battered towards each other and braced on all four faces by rigid diagonal bracing with riveted connections. The four columns shall be connected together at the top by a rigid frame of sufficient dimensions and strength to support the bedplates of the main girders; the bottom of the tower shall be braced by struts, both longitudinally and transversely, of sufficient strength and stiffness to overcome the friction of the base plates due to temperature movements.

Suitable intermediate cross frames between the four columns shall be used where necessary to keep the tower square and in shape.

Stability.

72. It is, generally, desirable that the tower shall have sufficient width at the base, both longitudinally and transversely, to prevent overturning by the assumed wind pressures, centrifugal forces, momentum of the train, and temperature, without depending upon the anchorage of the masonry pedestals under the legs. This object may, generally,

be obtained by making the transverse width at the base one-third of the height in addition to the width at the top of the tower, and the longitudinal width at the base one-sixth of the height in addition to the width at the top.

Anchor bolts shall be provided of sufficient strength to utilise the weight of the masonry pedestals, as an additional safeguard against overturning by excessive lateral pressures.

Diagonal Bracing.

73. The transverse diagonal bracing shall be of sufficient strength to resist the combined stresses due to wind and centrifugal force.

The longitudinal diagonal bracing shall be of sufficient strength to resist the combined stresses due to the momentum of the train and the friction of sliding bedplates; or a longitudinal wind pressure of three-fourths of the transverse wind load, added to the friction of the dead load on the girder bedplates.

Columns.

74. The columns or legs shall be of sufficient strength to resist the vertical components of the stresses due to the wind and centrifugal forces, momentum and temperature, in addition to the dead and live loads.

Generally, if sufficient sectional area is provided in the legs to resist the dead, live and wind loads, and centrifugal forces where the bridge is on a curve, it is not necessary to increase the sectional area for the stresses due to the momentum of the train, temperature and end wind pressure, as such a combination of stresses is improbable.

Anchorage.

75. In computing the greatest tension in the legs or anchor bolts, the calculation shall be made for both the loaded and unloaded structures. In double-track structures, a train of empty cars shall be placed upon the leeward track.

Anchorage shall be provided equal, at least, to twice the maximum uplifting force.

Joints.

76. For convenience in erection, the joints in the tower legs shall be made immediately above the panel joints. The whole section of the leg may be cut through and the abutting ends machined to a good bearing. The joints shall be spliced to transmit the whole of the load as a shearing stress through the rivets.

Swing Bridges.

Type.

77. The class of swing bridge shall be the centre bearing or rim bearing type.

Centre Bearing Type.

78. In swing bridges of the centre bearing type with rollers at the tail end, an excess of counter-weight shall be provided so that the tail rollers shall have a load of at least 10 tons on them when the bridge is loaded with 12 in. of snow, and the timber decking taken at 10 per cent. heavier to allow for saturation with moisture. The rollers shall be of cast steel, and shall not be less than 12 in. in diameter and 5 in. wide on the tread. The roller path shall be of cast iron or wrought steel of sufficient width to distribute the load on the masonry, and shall be secured to it by 1¼-in. diameter fullered bolts at least 8 in. long, spaced about 18 in. apart alternately.

Rim Bearing Type.

79. The turntable shall be of the latest improved type and of sufficient strength and rigidity to maintain its form and distribute the load over the masonry. The whole turntable shall be erected in the shop, previous to despatching to the site.

Drum Girder. 80. The drum girder shall be of sufficient strength and stiffness to distribute the load upon the rollers. It shall be true to dimensions and be prepared to receive the upper roller path. The web plates shall be of such thickness that the rivets supporting the lower flange angles shall have sufficient bearing to distribute the load from one roller to the web plates in a distance equal to its diameter.

In heavy bridges no material less than ½ in. in thickness shall be used and in light bridges ⅜ in. in thickness. The connection to the main girders of the bridge shall be designed of ample strength to resist the shock in starting and stopping the turning motion. The girder shall be proportioned as for plate girders.

Roller Paths. 81. The upper and lower paths shall be of cast steel, machined and finished true to dimensions, at joints and on bearing surfaces. The drum girder shall be adjusted to the upper path by steel folding wedges about 30 in. apart and afterwards drilled for 1-in. diameter bolts. After erection at the site, the spaces between the drum girder and the upper path shall be filled solid with rust cement well stemmed in. The lower roller path shall have suitable cover plates at the joints and shall be secured to the masonry by 1¼-in. diameter fullered bolts 8 in. long and about 18 in. apart alternately.

Rollers. 82. The rollers shall generally be of solid forged steel secured to the inner and outer spacing rings by bolts passing through their centres. Suitable gun-metal liners and collars shall be provided at all bearing surfaces and provision shall be made at the ends of the rollers for adjustment. The rollers and the surfaces of the upper and lower paths shall be turned to form parts of conical surfaces with a common vertex at the centre.

Roller Frame. 83. All radial bars and the roller frame of the live ring shall be formed of rigid members with suitable tangential bars to maintain the relative motion of the parts of the frame.

Centre Casting. 84. The centre casting shall have ample strength to centre the bridge and to prevent its displacement from objects striking the bridge.

Rack Segments. 85. The rack shall be of cast steel in short lengths and shall be planed on the bearing surfaces and joints, and connected to the lower path with fitted lug bolts.

Racks and Gears. 86. The cast racks and gears shall have the pitch lines of teeth exactly in the same plane. Gears shall be true to bevel, truly circular and bored at right angles to plane of action, to fit the shafts accurately. The teeth in gears shall be machine cut, where specified, to exact pitch and in all cases shall mesh accurately. The hubs of gears shall be machined.

End Lifts. 87. Effective end lifts shall be provided which shall give the necessary reaction in the shortest time to prevent the ends of the bridge being raised by a load on the opposite arm.

Turning Machinery. 88. Efficient power machinery shall be provided to open and close the bridge in the specified time. Hand-power machinery shall also be provided.

Lubrication. 89. Suitable means of lubrication shall be provided for all bearing surfaces in motion.

Working Stresses. 90. The unit working stresses in main girders shall be 10 per cent. less than those for simply supported spans. The length of the long arm shall be the measure of the span for comparison

Road Bridges.

Loading.

Dead Loads. 91. The dead load shall consist of the whole weight of the steel superstructure and any cast steel or ironwork, the materials forming the roadways, channels, kerbs, footways and handrails and the tramway tracks, water, gas and electric mains, if such exist upon the bridge.

The permanent way for a tramway track shall be assumed to weigh 100 lb. per lineal foot per track.

For spans of less than 200 ft., the total dead load shall be assumed to act at the loaded chord. For spans of 200 ft. and over, the total dead load shall be distributed at top and bottom chords as in paragraph 9 of Railway Bridge Superstructures.

Live Loads. 92. For the purpose of estimating the probable live loads, the bridges shall be divided into three classes as follows:—

Class I.—Roads with light traffic, such as branch country roads.

Class II.—Roads with occasional heavy traffic, such as main country roads.

Class III.—Roads with continued heavy traffic, such as in large towns and in manufacturing districts.

The following live loads shall be assumed:—

(*a*) *Pedestrian Traffic. Roadway.*

Class I.—For the main trusses. 84 lb. per square foot of road surface for all spans up to 100 ft., and reduced by 5 per cent. for each increase in the span of 20 ft., but in no case to be less than 56 lb. per square foot.

For the floor girders. 84 lb. per square foot for all spans.

Class II.—For the main trusses and floor girders.

Increase Class I. by 25 per cent.

Class III.—For the main trusses and floor girders.

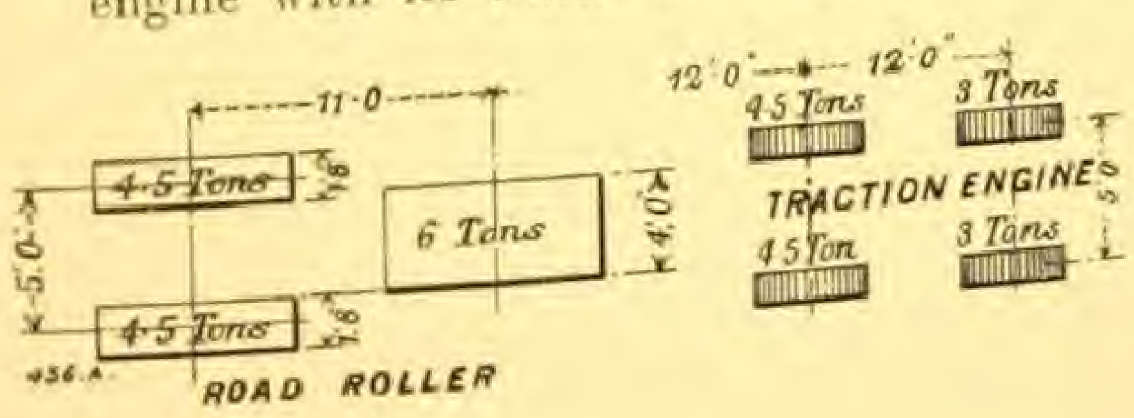
Increase Class I. by 50 per cent.

All classes.—Pedestrian traffic. Footways. To be the same as for roadway, but in no case to exceed 84 lb. per square foot.

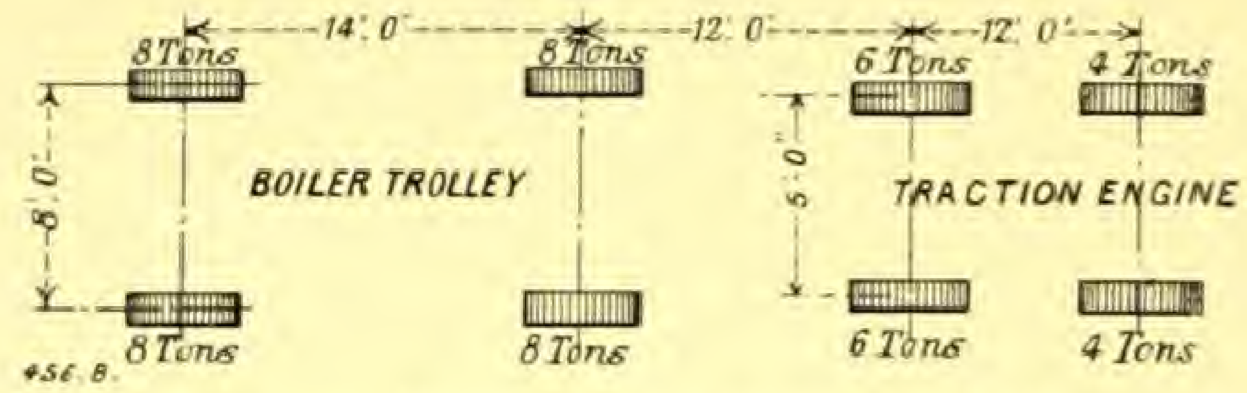
(*b*) *Vehicular Traffic.*

Class I.—A vehicle weighing 4 tons on two axles 6-ft. centres and 5-ft. gauge and occupying a width of 8 ft.

Class II.—A road roller or a traction engine, weighing 15 tons and occupying a width of 10 ft., followed by three loaded trucks weighing $8\frac{1}{2}$ tons each and averaging 10 cwt. per foot run. Only one engine with its trucks to occupy the bridge at one time.



Class III.—A road roller, as in Class II., or a traction engine weighing 20 tons and occupying a width of 10 ft, followed by a loaded boiler trolley weighing 32 tons. Only one engine and trolley to occupy the bridge at the same time.



Electric Trams.

93. The following loads per lineal foot per track shall be allowed for tramways The car is assumed to weigh 16 tons, and to be 25 ft. long on two axles 7-ft. centres by 4-ft. $8\frac{1}{2}$-in. gauge, and occupying a width of 10 ft. :—

10 cwt. per lineal foot plus an excess load of 8 tons anywhere on the track for a span of 20 ft., the load per lineal foot being reduced by $\frac{1}{4}$ cwt. for each increase in the span of 10 ft., but in no case to be less than $7\frac{1}{2}$ cwt. per lineal foot.

The live load stresses shall be the maximum stresses produced by the pedestrian and vehicular or tramway traffic taken together or separately and considered as stationary or as moving in either direction.

In estimating the stresses due to the combined pedestrian and vehicular or tramway traffic, the width of roadway occupied by the vehicular traffic shall be considered as occupied by the pedestrian traffic.

Unless otherwise specified, the tramway tracks shall be assumed to occupy the centre of the bridge in the case of a single track, and double tracks shall be placed symmetrically about the centre line of the roadway, with a distance of 10 ft. between their centres.

Vehicles and traction engines, with their following loads, shall be assumed to occupy any part of the roadway, and in calculating the stresses in the trusses their centre lines shall be assumed to be not less than 5 ft. from the inside face of the main girders.

Traction engines and tramways shall not be assumed to occupy the bridge at the same time unless the width of roadway exceeds 30 ft.

Unless tramways are specified, the bridge shall be designed for traction engines and other vehicles.

Wind-Pressure.

94. The wind shall be assumed acting in either direction, horizontally, and to be blowing at a slight angle to the axis of the bridge, so as to take effect on the exposed areas of the floor and of both windward and leeward girders, except when the latter is temporarily screened by a passing vehicle. One-half of the exposed surface of the leeward girder shall be included in the total area acted upon by the full wind-pressure, except when the distance between the main girders is more than twice their depth, when the whole exposed area of leeward girder shall be taken.

With no vehicles or live load on the bridge, the wind-pressure shall be assumed to be 50 lb. per square foot; and with the live load on the bridge, at 30 lb. per square foot of exposed surfaces of vehicles and bridge. The vehicles shall be assumed to cover the whole length of the bridge and be taken at 8 square feet per lineal foot, and the centre of pressure of the surface at 6 ft. above floor level, except in the case of trams, where the surface shall be taken at 14 square feet per lineal foot, and the centre of pressure at 8 ft. above rail level. The wind-pressure on the vehicles shall be treated as a moving load.

The maximum stresses resulting from either condition to be taken in determining the necessary sectional areas of the parts.

In providing the necessary anchorage for the structure, the bridge shall be assumed to be covered with a train of empty vehicles weighing 6 cwt. per lineal foot, or tram cars weighing 10 cwt. per lineal foot, and the live load placed on the leeward side of the bridge.

Momentum

95. Special attention shall be given to the details of the structure to provide for the longitudinal stresses resulting from the tractive force of the trams, or from the sudden application of brakes to the tram while on the bridge, and the horizontal force resulting from such action shall be taken as one-fifth of the weight of the tram.

Centrifugal Force.

96. When a bridge is on a curve, the resulting horizontal stresses due to the centrifugal action of the rolling load shall be provided for.

In summating the resulting stresses, the wind shall be assumed to be acting in the same direction as the centrifugal force.

Working Stresses.

Impact.

97. To provide for the effects of the dynamic action of the live load on lightly loaded girders or members of girders, the live load stresses shall be increased by 25 per cent. in the case of the main girders and $33\frac{1}{3}$ per cent. in the case of the floor girders.

Permissible Unit Stresses.

98. All bridgework and trestle piers shall comply with the whole of the following conditions:—

1. The combined stresses resulting from the live load, dead load, wind, momentum and centrifugal forces, shall not produce a greater tensile stress than one-half of the elastic limit, or equal to three-tenths of the minimum ultimate tensile strength of the material, nor more than the corresponding compressive, shearing, bearing and bending stresses as hereinafter set forth; but—
2. The combined stresses resulting from the rolling load and dead load alone, exclusive of wind, momentum and centrifugal force, shall not produce greater tensile stresses than those tabulated below.

Tensile Stresses, Net Section.

99. For main girders, cross girders and stringers—

Bottom chords	7 tons per square inch
Diagonals	5 to 7 " "
For wind-bracing	$8\frac{1}{2}$ " "
For floor suspenders	$3\frac{1}{2}$ " "

Note.—The 5 tons stress on the diagonals will apply to those at the centre portion of the span, and to the counter-bracing at the same point. The higher stresses will apply to those at the end portions of the span, where the variations of stress are not so great. Intermediate diagonals will be subject to stresses lying between the two limits.

Compressive Stresses, Gross Section.

100. For plate girders, the gross sectional area of the compressive flange shall not be less than that of the tension flange, nor shall the compressive stresses per square inch be more than 85 per cent. of the corresponding specified tensile stress.

For truss or lattice girders, the compressive stress per square inch shall in the case of riveted members not exceed the fraction $(0.95 - 0.003\,r)$ of the corresponding specified tensile stress; nor, in the case of pin-connected members, exceed the fraction $(0.95 - 0.0045\,r)$, where r is the ratio of the length of the unbraced portion of a member to its least radius of gyration; nor in any case shall it exceed 85 per cent. of the said tensile stress. No compression member shall have a greater length than 120 times its least radius of gyration, except for wind-bracing, where it may have a length not exceeding 140 times its least radius of gyration.

Alternating Stresses.

101. Members subject to alternate tension and compression shall be proportioned as struts and have sectional areas equal to the greater area added to one half of the lesser area required for the compressive and tensile stresses considered independently, except in the case of wind-bracing, where the sectional area may be proportioned to resist the greater stress.

Shearing, Bearing and Bending Stresses.

102. The shearing, bearing and bending stresses per square inch shall not exceed the following limits:—

(1.) *In Truss or Lattice Girders, and all Web or Flange Joints in Plate Girders.*

(*a*) For machine-driven rivets and turned bolts or pins of a driving fit

Shearing stress	...	$\frac{3}{4}$ of the permissible maximum tensile stress per square inch in the girder or member.
Bearing stress	...	$1\frac{1}{2}$ ditto
Bending stress	...	$1\frac{1}{2}$ ditto

(*b*) For hand-driven rivets having a length over four diameters.
The number found for (*a*) to be increased by 10 per cent.

(*c*) For ordinary black bolts.
The number found for (*a*) to be increased by 25 per cent.

(2.) *In Plate Girders.*

Shearing stress in rivets	...	$\frac{7}{8}$ of the permissible maximum tensile stress per square inch in the girder.
Shearing stress in web plate	...	$\frac{1}{2}$ ditto
Bearing stress on rivets	...	$1\frac{3}{4}$ ditto

(3.) *Bending Stress on Members Subject to Direct Tensile or Compressive Stresses.*

Where such stresses occur the member shall be proportioned to the algebraic sum of the stresses resulting from the direct stresses and three-fourths of the maximum bending stress, and the stress per square inch shall not exceed that permitted for the direct stresses. The member shall be considered as a beam freely supported at the ends, and the bending moment at the ends shall be considered equal to that in the centre but in opposite direction.

Rollers and Bedplates.

103. The pressure in pounds per lineal inch on rollers of rolled steel shall not exceed $300\,d$, where d equals the diameter of the rollers in inches; in the case of live rollers under swing bridges, the pressure shall not exceed three-fourths of this amount. Bed-plates and rockers shall be of sufficient area and strength to distribute the load over the masonry without exceeding a pressure of 16 tons per square foot for hard stones, of 20 tons per square foot for granite, and 10 tons per square foot for cement concrete (4—2—1).

Wrought Iron-work.

104. Where wrought-iron is used for any girder or member of a bridge, the working stresses shall be 80 per cent. of those specified in the case of steel for members subject to tensile and bending stresses, and for short compression members; 85 per cent. for long compression members, and 90 per cent. for members subject to shearing and bearing stresses.

Wind and Centrifugal Force.

105. Where the stresses resulting from the dead and live loads are combined with those due to the wind alone, or with the wind and centrifugal force, the preceding working stresses per square inch may be increased 25 per cent., but in no case shall they exceed three-tenths of the minimum ultimate tensile strength of the material.

Opening Bridges.

106. The unit stresses on the main girders of opening bridges shall be reduced by 10 per cent.

Connections.

107. Connections shall be proportioned to develop the full strength of the member notwithstanding that the calculated stress may be less.

Structural Details, Etc.

108. The paragraphs on pages 432 to 434 of the Railway Bridge Specification shall apply generally to road bridges unless otherwise specified. The types of bridges suitable for particular spans may be modified, and a minimum weight of 4 lb. per lineal foot allowed for shapes, and a minimum thickness of ¼ in. for thicknesses of plates or bars. Suitable parapets shall be provided having a height above the footways of at least 4 ft., and capable of resisting a horizontal pressure of about 56 lb. per lineal foot applied at the coping level.

Types of Bridge Floors.

Timber Floors.

109. When timber floors are specified, the roadway planks shall be at least 3 in. in thickness, nor less than one-twelfth of the distance between the longitudinal joists or stringers, and from 8 in. to 10 in. in width. They shall be laid transversely with ¼-in. open spaces.

When an additional wearing surface is used for the roadway, it shall be 1½ in. thick and not exceeding 6 in. in width, laid transversely with close joints. The bottom course shall have a minimum thickness of 2½ in. and laid diagonally with ½-in. open spaces.

When wooden joists are used to carry the planking, they shall not be less than 3 in. in thickness, nor greater in depth than four times their thickness. They shall be spaced not more than 2½ ft. apart centres, and shall be checked over their seat angles or floor girders by at least ½ in., and the tops brought to exact level before the planks are fixed. The intermediate joists shall, preferably, lap by each other so as to extend over the full width of the floor girder, and shall be separated by ½ in. to allow a free circulation of air. The outer joists shall abut so as to provide flush surfaces from end to end of the span.

The wooden joists shall be securely fastened to the floor girders by hook bolts ⅝ in. in diameter. The bottom course of planking shall be spiked to each joist on which it rests by two 7-in. by ⅜-in. cut spikes, or wire nails in each plank, and the top wearing surface nailed to the planking.

Where wooden joists are not used, a wooden sleeper of not less than 9 in. in width

and 4 in. in thickness shall be securely fastened to the top flange of the floor girder by coach screws ⅝ in. in diameter, and spaced not more than 18 in. apart alternately. The planking shall be spiked to the sleeper.

Wheel guards of a cross section not less than 4 in. deep by 6 in. wide shall be provided on each side of the roadway. They shall be blocked up from the flooring by blocks at least 12 in. long by 6 in. wide and 2 in. thick, and spaced not more than 7 ft. apart centres. These blocks are fastened to the floor by four 4½-in. cut spikes. The guard timbers are held in place by a bolt through the centre of each block ¾ in. in diameter and passing through the joist beneath. The guard timbers shall be spliced over a blocking piece with a lap joint at least 6 in. long. The guard timbers shall be protected at their upper corner next the roadway by steel angles at least 3 in. by 3 in. by ¼ in., securely fastened by countersunk screws about 18 in. apart.

The footway planks shall be 2 in. thick and about 6 in. wide, laid with ½-in. open spaces. Each plank shall be spiked to the joist or sleeper by two 6-in. cut spikes.

All planks shall be laid with the heart side down, and shall have full and even bearings and be firmly attached to the stringers.

Solid Floors. 110. The materials forming the wearing surface of the roadway shall be laid upon a bed of gravel concrete of a thickness of at least 2½ in. over the highest point of the floor girders to be covered, excepting rivet and bolt heads. The concrete bed shall be cambered at least 1 in. for every 5 ft. in width of the roadway to provide suitable transverse drainage.

The kerb shall be formed of solid granite 12 in. wide by 8 in. deep, projecting not less than 4 in., and not more than 6 in. above the channel.

The channels may be formed either of solid granite 12 in. wide by 6 in. deep, or of three or more parallel courses of granite setts, each 6 in. deep by 4 in. wide. The channels shall have a fall of at least 1 in 50 towards gulley gratings.

Suitable gulleys shall be provided to drain the channels clear of all parts of the metal work.

The materials forming the wearing surface of the footpaths shall be laid upon a bed of gravel concrete of a thickness of at least 1½ in. over the highest point of the footway girders, excepting rivet and bolt heads. The concrete bed shall have a fall towards the channels of about 1 in 72 to provide suitable drainage.

Where ordinary macadam is used for the roadway, it shall have a minimum finished thickness of 6 in., tar macadam of 4 in., and asphalte of 1½ in. for light traffic and 2 in. for heavy traffic. Granite setts may be 7 in. long by 5 in. deep, or 6 in. by 4 in., or 5 in. by 3 in. Soft-wood paving blocks are generally 9 in. long by 6 in. deep by 3 in. wide, and hardwood paving blocks 9 in. by 3 in. by 5 in. deep.

Where asphalte is used for the footpaths, it shall have a minimum finished thickness of not less than ¾ in., cement of 1 in., and tar paving of 2 in. Stone paving slabs are generally 2 in. thick.

When wood block paving is used for the roadway, it may rest on a timber floor of from 4 in. to 5 in. in thickness, which is securely bolted to the floor girders.

Steel Plate Floors. 111. The steel plates for carrying the materials forming the road and footway may be either flat, buckled, curved or corrugated.

Flat Plates. 112. The flat plates shall be riveted on all edges to the floor girders or intermediate supports, and shall be suitably stiffened at intermediate points. The plates shall have a

bearing of not less than $2\frac{1}{2}$ in. at their edges. The rivets shall not be less than $\frac{3}{4}$ in. in diameter and about 6 in. pitch.

Buckled Plates.

113. The buckled plates shall be riveted on all edges to the floor girders or intermediate supports, and shall have a flat bearing strip all round of not less than $2\frac{1}{2}$ in. in width. The rivets shall not be less than $\frac{3}{4}$ in. in diameter and about 6 in. pitch.

The plates shall have a camber of not less than 2 in., nor less than one-twentieth of the greatest clear distance between the supports.

Curved Plates.

114. The curved plates shall generally be placed across the bridge and supported by the cross girders, or, if laid longitudinally, stringers must be used for supporting them. They shall have a flat bearing of not less than $2\frac{1}{2}$ in. along their longitudinal edges for riveting to the floor girders, and shall be riveted at the ends to the web plates of the main girders, or to a ballast plate by means of an angle bar. The rivets shall not be less then $\frac{3}{4}$ in. in diameter and about 6-in. pitch.

The plates shall have a camber of not less than 3 in., nor less than one-fifteenth of the clear distance between the supports.

Provision shall be made to prevent the plates spreading under the loads.

The plates shall be suitably stiffened by suitable angle or tee bars riveted to the underside.

Corrugated Plates.

115. The corrugated plates shall be laid either between the main girders or supported on the cross girders. They shall preferably be formed so that the top section of one corrugation laps over the top section of the adjoining corrugation by at least 3 in., and is riveted to it by one line of rivets not less than $\frac{3}{4}$-in. in diameter and not more than 4-in. pitch.

The corrugated plates shall be riveted to their supports, and where they abut upon a web plate, a cleat shall be riveted to the top of each corrugation and to the web plate.

The corrugated plates shall not have a less depth than one-twentieth of the clear distance between the supports.

Minimum Thickness of Floor Plates.

116. The steel plates in the floors shall not be less in thickness than $\frac{1}{4}$ in. under the roadway or footways.

Jack Arches.

117. Jack arches shall be formed of one-half brick ring of brindle bricks in cement or Staffordshire blue bricks in cement. The skewbacks shall be formed of one part of cement to two parts of clean sand and laid on the bottom flange of the girders to the correct angle to receive the brick arch ring. The spandrils shall be filled in with cement concrete (6 to 1) to a level of about $1\frac{1}{2}$ in. over the extrados at the crown.

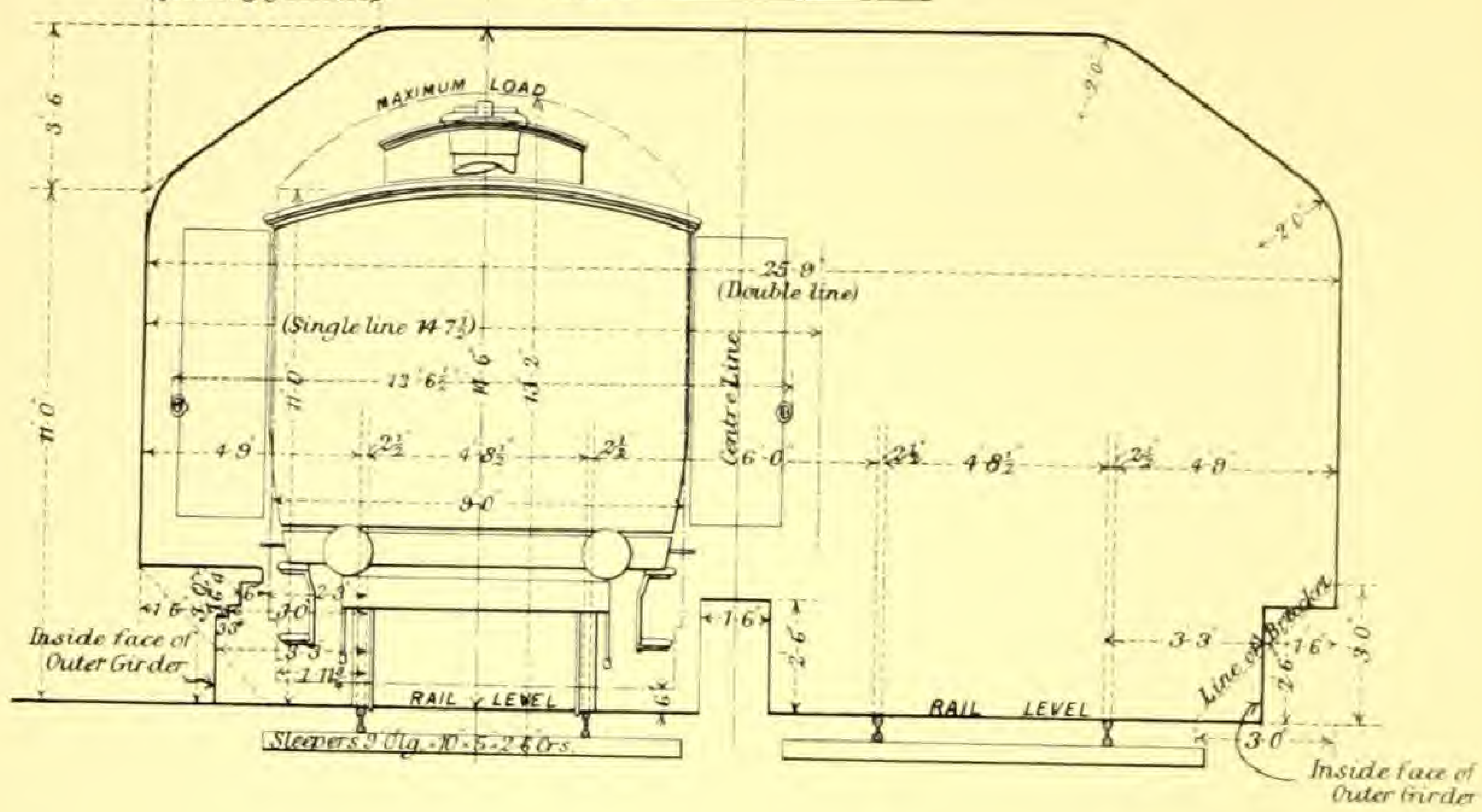
The rise given to the arch shall be such that the extrados at the crown shall be about level with the top flange of the girders supporting it, but shall not be less than one-fifth of the clear span.

Tie bolts, of not less than 1-in. diameter and not farther than 5 ft. apart, shall be used to prevent the arch spreading.

To prevent moisture percolating through the jack arch rings, $1\frac{1}{2}$ in. of asphalte shall be spread over the whole surface of the roadway, and shall be laid in two layers of $\frac{3}{4}$ in. and lapped up at the kerbs about 2 in.

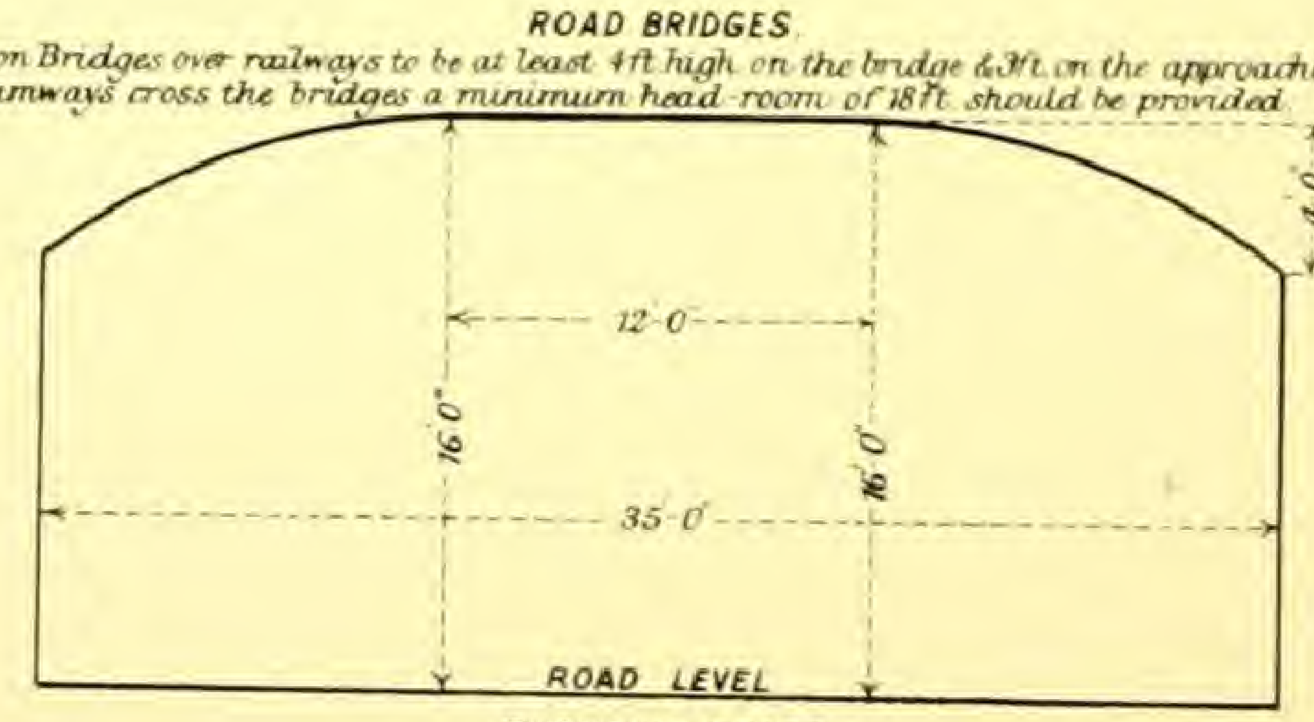
Standard Clearances for Bridges.

RAILWAY BRIDGES.
DOUBLE LINE BRIDGES 26 FEET WIDE.
SINGLE D° 15 D°

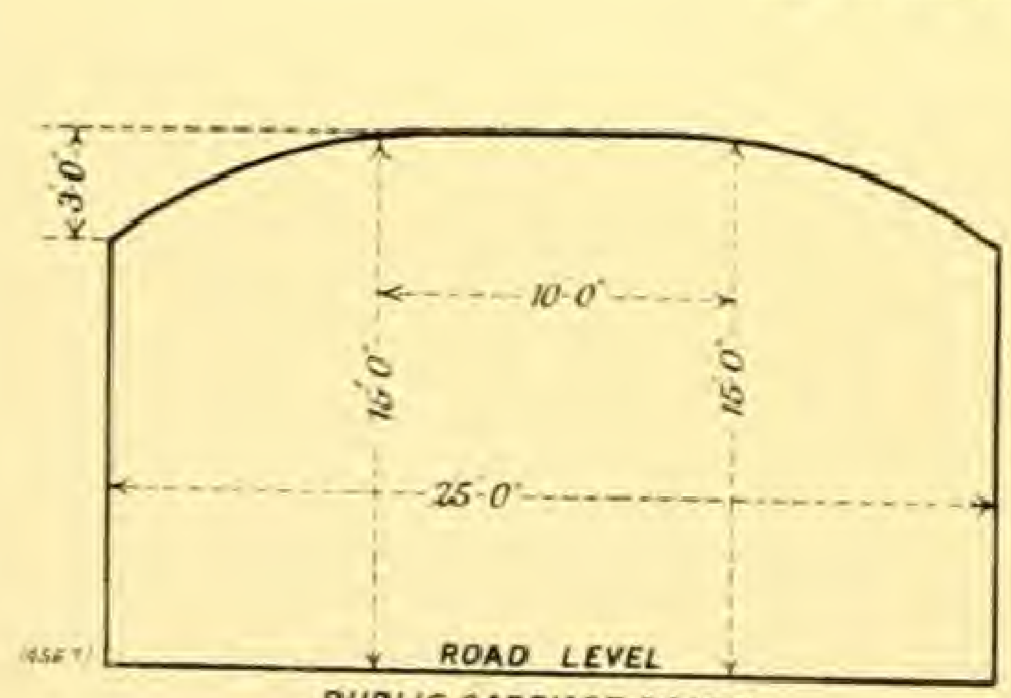


ROAD BRIDGES.

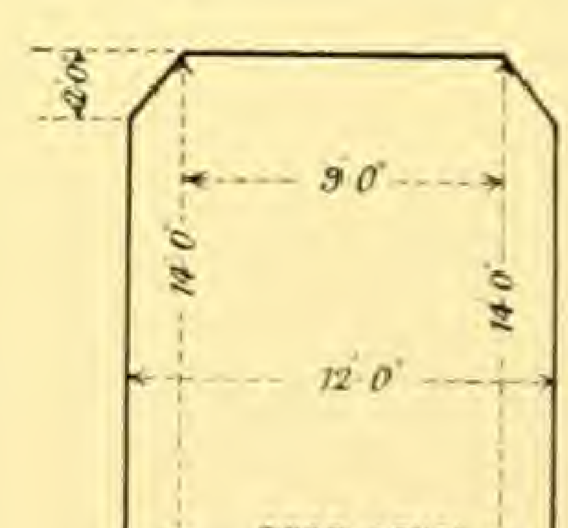
NOTE. Parapets on Bridges over railways to be at least 4ft high on the bridge & 3ft on the approaches to it. Where Tramways cross the bridges a minimum head-room of 18 ft should be provided.



TURNPIKE ROADS.
Max. Gradient 1 in 30.



PUBLIC CARRIAGE ROADS.
Max. Gradient 1 in 25.



PRIVATE OR OCCUPATION ROADS.
Max. Gradient 1 in 16.

General Specification for Workshop Buildings.

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General Specification for Workshop Buildings.

MATERIALS.

General.

1. The whole of the structural portion, except the tee astragals and ventilator standards, shall be of open-hearth rolled steel. It shall be free from laminations or surface defects, and shall comply with the specification of the British Engineering Standards Committee for structural steel.

The tee astragals shall be generally of wrought iron, and the ventilator standards of cast iron.

Rolled Steel.

2. Strips cut lengthwise or crosswise shall have an ultimate tensile strength of not less than 27 tons, nor more than 32 tons per square inch of original section, with an elongation of at least 20 per cent. in a length of 8 in.

Rivet Steel.

3. Rivet steel shall have an ultimate tensile strength of not less than 26 tons, nor more than 30 tons per square inch of original section, with an elongation of at least 25 per cent. in a length of 8 diameters.

Cast Iron.

4. All castings shall be of the best tough grey metal of such a strength that a bar 1 in. thick by 2 in. deep placed upon bearings 3 ft. apart will sustain without fracture a weight of 27 cwt. placed at the centre with a deflection of not less than $\frac{1}{3}$ in.

Wrought Iron.

5. The wrought iron used for bolts or bars under 2 in. in diameter shall have an ultimate strength of not less than 23 tons per square inch of original section, with an elongation of not less than 15 per cent. in 8 in., and shall bend double when cold without cracking.

Timber.

6. All timber shall be of the best quality, sawn true and out of wind, dressed where specified, of full size and free from shakes, large or loose knots, decayed wood, sap, worm holes, or other defect which would impair its strength or durability.

All external exposed woodwork, except weather-boarding, shall be of red pine or equal. Weather-boarding shall be of white pine. Where the woodwork is not exposed to the weather it shall be of white pine or similar quality, except the main horizontal beams in timber framing, which shall be of pitch pine or other approved timber. Timber shall not be dressed unless specified.

Galvanised Sheeting.

7. The corrugated sheeting shall be of first-class quality and well galvanised with at least 3 oz. of spelter per square foot. Unless otherwise specified it shall have a thickness of No. 18 W. G. before galvanising.

Flat sheets shall be galvanised, and, unless otherwise specified, shall have a thickness of No. 16 W. G. before galvanising.

Painting.

8. The whole of the structural work before leaving the shop shall be scraped clean and receive one coat of the best oxide paint.

Where two surfaces come in contact, one of them shall receive one coat of paint, and all parts which are not accessible to painting after erection shall receive two

coats of paint before being riveted together. After erection at the site the whole shall receive one finishing coat of an approved colour.

The parts of columns and other steelwork which come in contact with brickwork, masonry, or concrete shall not be painted or oiled, but shall be scraped free from rust, and shall receive a coat of liquid cement applied with a brush as the steelwork is being built in. This cement wash shall be applied by the contractor for the brickwork, masonry, or concrete.

Galvanised sheeting shall not be painted unless specified.

All woodwork in gates shall receive two coats of paint, one before and one after erection.

All woodwork in roof gangways shall receive a coat of tar after erection. It shall not be creosoted unless specified.

Glass. 9. The glass on the roof slopes shall be of the best quality of rough cast British plate, $\frac{1}{4}$ in. in thickness.

For vertical side glazing, the glass shall be of the best quality of rough cast British plate, $\frac{3}{16}$ in. in thickness.

The glazing for windows shall be of the best quality British rolled plate, $\frac{1}{8}$ in. in thickness where it is desired to have obscured glazing, and of 21 oz. sheet glass where it is desired to have clear glazing.

All glazing, unless otherwise specified, shall be bedded in putty made with the best whiting and linseed oil, well beaten and kneaded into a stiff paste.

Slates. 10. Slates shall be of the best quality obtainable in the locality, and shall be properly sorted and squared.

Lead Flashings. 11. Where sheet lead is used for ridges and apron flashings, it shall weigh not less than 4 lb. per square foot, and for skews liable to be used for walking upon it shall be 6 lb. per square foot.

Water. 12. It is assumed that the proprietors will supply clean water free of charge, and lay a pipe to a convenient point on the site.

Sand. 13. All sand shall be of large, rough and sharp quality, entirely free from earthy matter or other impurities.

Cement. 14. All cement shall be the best fresh Portland cement of a weight, fineness and quality to comply with the specifications of the British Engineering Standards Committee.

Bricks. 15. The bricks for walling shall be uniform in colour, machine made, hard, sound, square and thoroughly kiln burned, and generally of a height to build three courses to the foot. Wire-cut, soft, bully or kiln-cracked bricks will not be used, nor half or broken bricks where whole bricks can be employed.

Mortar. 16. (*a*) The mortar for building shall be composed of one part of Portland cement to three parts of sand by measure, and for pointing and plastering it shall be composed of equal parts of Portland cement and sand.

(*b*) Where specified, the mortar may be composed of well-slaked lime and clean, sharp river sand, entirely free from salt or other impurities, in the proportion of one part by measure of clot lime to three parts by measure of sand, all properly mixed and soured up before the work is commenced.

Concrete.

17. The concrete for wall and column foundations shall be composed by measure in the dry state of one part of Portland cement to two parts of sand and four parts of bricks broken to pass a 2-in. ring, or one part of Portland cement to six parts of river gravel containing a suitable proportion of sand. The whole of the materials shall be prepared on timber platforms and turned over twice while dry, after which sufficient water shall be added and the mass turned over until completely incorporated. The concrete shall be laid in the trenches and properly punned until it shakes like a jelly.

Loading.

18. The structure shall be designed to withstand the following loads :—

Dead Loads.

19. In determining the sections of any truss, girder, or column, the total dead weight coming on such shall be taken into account.

Live Loads.

20 These shall include the maximum loads from all travelling, jib or mono cranes, including their own weight and the pull from belting.

(*a*) Provision shall be made for the effect of the sudden application of brakes to the rapidly moving crane, either in longitudinal or cross traverse, and the forces due to acceleration and retardation. The horizontal longitudinal braking force shall be taken at not less than one-eighth of the load on the driving wheels and divided between the rails in proportion to the load carried by each. The horizontal force due to the cross travel of the loaded crab shall be taken at one-tenth of the weight of crab and load.

(*b*) Provision shall be made for the forces due to the dragging weights along or across the floor by means of the cranes, which shall be assumed to be a horizontal force at least equal to one-tenth of the lifting capacity of the crane.

(*c*) The pull of belts and the weights of pulleys and shafting shall be taken as an additional load in hundredweights per lineal foot at least equal to $\frac{D^2}{8}$, where D is the diameter of the shaft in inches. It shall be assumed to act at an angle of 45 degrees.

Wind Pressure.

21. The building shall be designed to resist an average steady horizontal wind pressure of 30 lb. per square foot of exposed surface. This value shall be taken for all members supporting an area of 300 square feet and under, and shall be diminished by 1 lb. per square foot for each 100 square feet of area in excess of this amount, but no value less than 20 lb. per square foot shall be used.

One-third of the wind pressure only shall be assumed to act with the dead load and maximum live loads.

Snow Load.

22. A load of 5 lb. per square foot of horizontal projection shall be allowed for on all surfaces where snow can collect. The snow load shall be taken in addition to the wind load.

Minimum Roof Loads.

23. No roof shall be designed for a less load than 28 lb. per square foot of horizontal projection for a glazed or sheeted roof and 35 lb. for a slated roof.

The roof loads shall be taken at the following weights per square foot of horizontal projection :—

Glazed Covering.

Dead Load.

	lb.
$\frac{1}{4}$-in. rough cast glass	3.50
Astragals ...	1.75
Putty Fillets	0.50
Purlins ...	2.00
Total dead load	7.75

Occasional Loads.

30 lb. wind pressure. Vertical component	20.00
Snow ...	5.00

Galvanised Corrugated Iron Covering.

Dead Load

	lb.
No. 18 W.G. galvanised corrugated sheeting	2.66
Rivets and straps ...	1.20
Purlins ...	2.00
Total dead load	5.86

Occasional Loads.

30 lb. wind pressure. Vertical component	20.00
Snow ...	5.00

Slates on Boarding.

Dead Load.

	lb.
Slating, 16 in. × 8 in.	5.60
Felt ...	0.25
Boarding, 1 in. thick	3.50
Purlins ...	3.25
Total dead load	12.60

Occasional Loads.

30 lb. wind pressure. Vertical component	20.00
Snow ...	5.00

Tiles or Slates on **L** *Laths.*

Dead Load.

	lb.
Slating, 24 in. × 12 in.	9.00
Lead nails ...	0.25
Purlins ...	4.00
Total dead load	13.25

Occasional Loads.

30 lb. wind pressure. Vertical component	20.00
Snow ...	5.00

Working Stresses.

General.

24. In no case shall the stresses under any combination of loads exceed three-quarters of the elastic limit of the material, nor be greater than the following values :—

Tensile Stresses.

25. The tensile stress on the net section shall not exceed the following limits :—

For live loads $\frac{1}{5}$ of the ultimate strength = 6 tons per square inch.
For dead loads $\frac{1}{4}$ ditto = $7\frac{1}{2}$ ditto
For wind and
snow loads $\frac{1}{3}$ ditto = 10 ditto

The total sectional area of a member shall be the sum of the areas required for each of the different loadings enumerated above.

Compressive Stresses.

26. The compressive stress on the gross section shall not be greater than 85 per cent. of the corresponding tensile stress, nor be more than the ratio given by Fidler's tables for compression members. The top booms of crane girders shall be of sufficient width to resist the forces from the cross travel of the crabs or from dragging weights. The maximum unit stress shall be limited to a flange whose width is less than one-twentieth of its length ; where the flange is less in width than one-twentieth of its length, the stress shall be reduced by $\frac{3}{8}$ ton per square inch for each increase in length of five times the width.

No member in compression shall have a greater length than forty-five times its least width, or 120 times its least radius of gyration.

Main columns shall have a length not exceeding forty times their least width, and preferably one-twenty-fifth.

Shearing Stresses.

27. The shearing stress on web plates of girders shall not exceed one-half of the permissible tensile stress, and the web plate shall have stiffeners at the bearings, and at all points of concentrated loading, and at intermediate points, not further apart than the depth of the girder, where the stress in tons per square inch exceeds $\dfrac{T}{1 + \dfrac{H^2}{3000}}$, where H = the distance between flange angles or stiffeners divided by the thickness of the web plate, and T = the permissible unit tensile stress in the girder.

The shearing stress on rivets or turned bolts of a driving fit shall not exceed seven-eighths of the permissible tensile stress in the case of plate girders and three-quarters in the case of lattice girders. For black bolts the number shall be increased by 25 per cent.

Bearing and Bending Stresses.

28. The bearing or bending stress on rivets or turned bolts of a driving fit shall not exceed one and three-quarter times the permissible tensile stress in the case of plate girders, and one and a-half times in the case of lattice girders. For black bolts the number shall be increased by 25 per cent.

Combined Bending and Direct Stresses.

29. Where members are subject to combined bending and direct stress they shall be proportioned for the algebraic sum of the stresses, and the permissible stresses may be increased by 10 per cent. The members shall be considered as partially continuous, and the bending moment equal to three-quarters of that due if the member were freely supported. The bending moment at the ends shall be assumed to be equal, but opposite to that in the centre.

Loads on Masonry.

30. The maximum pressures under the dead and live loads in masonry shall not exceed the following limits :—

Granite bedstones	25 tons per square foot.
Limestone ditto	20 ditto
Sandstone ditto	16 ditto
*Ashlar masonry in cement	15 per cent. less
*Coursed rubble ditto	30 ,,
*Random rubble ditto	60 ,,
*Brindled bricks in cement mortar ...	10 tons per square foot.
Cement concrete	10 ditto
*Stock brickwork in cement mortar ...	6 ditto
* Ditto in lime mortar ...	4 ditto

Under the dead and live load and wind pressure the permissible pressures may be increased by 25 per cent.

* *For Piers.*—This pressure shall be allowed where the height is not greater than twelve times the width.

Structural Details.

Minimum Sections.

31. Where direct stress is transmitted no steel shape shall weigh less than 4 lb. per lineal foot, nor be less in width than $2\frac{1}{4}$ in.; nor shall any structural member, plate or bar have a less thickness than $\frac{1}{4}$ in. Where the steelwork is subject to corrosive influences an addition of $\frac{1}{16}$ in. shall be made to the thickness, or a corresponding increase in sectional area over that required for the calculated stresses.

The pitch of the rivets in built members or girders shall not exceed 8 in., or be more than sixteen times the thickness of the thinnest outside plate or bar, nor be less than three diameters. The distance from the outside edge of a plate or bar shall not be less than one and a-half diameters. At the ends of built columns for a length equal twice the width of the column the pitch of rivets shall not exceed four and a-half diameters. The thickness of metal in compression shall not be less than one-sixteenth of the distance between supports in the line of stress, or be less than one-fortieth of the distance between supports at right angles to the line of stress, nor less than one-twelfth of the distance from the edge of the bar or plate to the line of support.

Lacing Bars.

32. Single lacing bars of columns shall have a thickness of not less than one-fortieth and double lattice bars connected by a rivet at their intersection of one-sixtieth of the distance between the rivets connecting them to the columns. The width of the lattice bars shall not be less than 3 diameters of the rivets passing through them, nor be less than one-eighth of the length of the bar. Angle bar lacing shall be used in all heavy columns. The lacing bars shall be connected to the main shafts of the columns by sufficient rivets to develop their strength as compression members. The distance between the lattice bars shall not exceed eight times the least width of the segments connected.

Tie Plates.

33. Where tie plates are used they shall have a length of not less than three-quarters of the width or depth of the columns and a minimum thickness of one-fiftieth of the distance between the line of rivets connecting them to the segments of the columns. Where stiffening angles are used on the horizontal edges of the tie plates, the thickness may be reduced.

Joints. 34. All joints shall be fully covered to transmit the loads as a shearing stress through the rivets. Cover plates shall have an area of at least 25 per cent. in excess of the section joined.

Riveted Connections. 35. The rivet area shall be sufficient to develop the full strength of the member, and at least two rivets shall be used in making connections, or one rivet in double shear, notwithstanding that the calculated stress may require less.

Purlins. 36. All purlins shall be made of simple shapes, designed as semi-continuous beams, and having a depth of not less than one-fortieth of their span. They shall be placed over the panel points of the roof trusses, otherwise the bending stress in the rafters must be provided for. Provision shall be made for the expansion of long lines of purlins by loosely bolting some of the joints.

Roof Trusses. 37. All roof trusses shall be constructed of rigid members capable of resisting tension or compression, and shall have a depth of not less than one-fourth of the span. A camber of at least 1 in. for every 40 ft. in length shall be given to all trusses.

Where there are travelling or jib cranes in a bay, the horizontal tie of the roof trusses between the columns shall generally be composed of two angle-bars, so as to act to some extent as a compression member.

Where shafting is supported from the roof-trusses the horizontal tie shall be constructed of suitable sections to permit of the shafting brackets or hangers being attached to it.

The trusses shall be rigidly secured to the columns by means of triangular brackets or knee braces.

Roof Girders. 38. Roof girders shall, as far as practicable, be of lattice construction with rigid members, and of ample depth to assist in bracing the shop either longitudinally or transversely. They shall preferably be placed between the main column shafts and not over them.

All girders over 40 ft. in length shall be given a camber of $\frac{3}{4}$ in. for every 40 ft. in length.

Crane Girders. 39. Crane girders shall generally be of plate construction, with a trough-shaped top flange of ample width (preferably one twenty-fifth of the length), and constructed to resist the vertical and lateral forces to which it may be subjected by the cranes.

The crane rail shall be riveted to the girder by rivets spaced about 18 in. alternate pitch. Bridge rails shall generally be used of the following weights:—For cranes up to 25 tons capacity a rail weighing 56 lb. per yard shall be used; for cranes up to 50 tons capacity a rail weighing 70 lb. per yard shall be used; and for cranes up to 100 tons capacity a rail weighing 106 lb. per yard shall be used. The rails shall not be included in the sectional area of the top flange.

The end plate shall be riveted to one end of the girder only, and provision made for bolting the abutting ends of the girders together by bolts spaced not more than 12 in. apart.

All girders shall be butted hard, end to end, and no provision made for expansion.

No camber shall be given to plate-webbed crane girders.

Columns. 40. Columns shall preferably be constructed so that a shaft is provided under each girder, but where the girders are carried by cantilever brackets from the columns the resulting bending stress shall be provided for.

All segments of columns connected by latticing shall have intermediate tie plates spaced not farther apart than five times the width of the column. The tie plates shall have a depth of not less than three-quarters the width of the column.

The long dimension of the base shall preferably be placed across the shop to assist the stability of the building.

All columns shall be secured to the concrete foundations by anchor bolts, placed as far apart as possible so as to facilitate the erection of the shops.

Framing for Sheeting.

41. The framing for supporting the corrugated sheeting shall generally be constructed of one or more horizontal lattice girders supported at the main columns, with intermediate horizontal angle bars supported at points not less than 7 ft. apart by vertical angle bars or girders fixed between the horizontal lattice girders.

The horizontal blades of the sheeting angles and girders shall be placed uppermost to support the hook bolts carrying the corrugated sheeting.

Diagonal Bracing.

42. Regard must be had in the design of the shops for the horizontal forces from the travelling jib and mono cranes, and from shafting or wind pressure. Shops over 20 ft. in height to the eaves shall have suitable diagonal bracing in a horizontal plane placed between the tops of the columns and secured to the roof ties to distribute the local forces from wind and cranes and to line and square up the building. To prevent the roof trusses being overturned by end wind pressure on the gables, the pair of roof trusses at each end shall be braced together between the rafters and in a vertical plane on the centre line. Vertical diagonal bracing shall be placed between the main columns in the sides and ends of the building to take up the horizontal forces due to cranes or wind pressure. Where there are deep roof girders this bracing may be omitted.

Ventilators.

43. Unless otherwise specified a continuous ventilator shall be provided only on the ridge of each roof. It shall be constructed with curved flashings at the top of the roof slopes, so that, as far as practicable, driving rain and snow will be kept out.

Weather Boarding.

44. The weather boarding shall have an average thickness of $\frac{3}{4}$ in., laid horizontally so as to lap over each other at least $\frac{1}{2}$ in. The maximum span for this boarding shall be 5 ft.

Timber Framing.

45. The framing for carrying the weather boarding shall consist of main horizontal timbers, trussed if necessary, between the columns, and spaced from 5 ft. to 10 ft. apart, according to circumstances. Vertical timbers spaced not more than 5 ft. apart shall be placed between the main horizontal timbers and let into them about $\frac{1}{2}$ in. at each end.

The depth of timber beams shall not be less than one-thirtieth of their span.

Steel Framing for Brick-work.

46. The steel framing for brickwork shall be formed of suitable braced girders or joists placed horizontally between the main columns, and with intermediate vertical girders or joists between them. They shall be placed so that the panels shall not exceed 150 square feet in area. Grouting holes shall be provided in the webs of the horizontal joists for grouting up the brickwork.

Gates.

47. All gates shall be placed outside the building, and hung by anti-friction pulleys from a suitable top runner secured to the framework or walls.

A cast-iron bottom guide of suitable section bedded in concrete shall be provided across the opening at the floor level, and extending each side of the opening for the full travel of the leaves of the gate.

The gates shall be at least 4 in. wider and 2 in. higher than the clear opening, and shall be made in two leaves when the opening is over 10 ft. One hinged wicket gate, 6 ft. by 2 ft., opening outwards, shall be provided in all gates.

A hood formed of $\frac{1}{8}$-in. steel plate shall be provided over gate openings for weather protection, and shall extend at least 6 in. each side of the opening.

Each gate shall be provided with a bolt, latch and padlock to secure the leaves, and a suitable lock fitted to the wicket.

The gates shall generally be made of dressed red pine, framed with stiles and rails, and lined with $\frac{7}{8}$-in. boarding, and shall not be less in thickness than $2\frac{1}{2}$ in. for gates $12\frac{1}{2}$ ft. square, 3 in. for gates 20 ft. square, and $3\frac{1}{2}$ in. for gates 25 ft. square.

Where steel gates are specified, they shall be made of braced angle frames and covered with galvanised corrugated iron of the same gauge as that used on the building or $\frac{1}{8}$-in. plating. Where the gates are in two leaves, one of the leaves shall have a flat strip riveted on the meeting rail, so as to cover the other leaf when the gate is closed.

The main framing angles shall not be less than $2\frac{1}{2}$ in. by $2\frac{1}{2}$ in. by $\frac{1}{4}$ in. for gates 10 ft. square, $2\frac{3}{4}$ in. by $2\frac{3}{4}$ in. by $\frac{5}{16}$ in. for gates 15 ft. square, 3 in. by 3 in. by $\frac{3}{8}$ in. for gates 20 ft. square, $3\frac{1}{2}$ in. by $3\frac{1}{2}$ in. by $\frac{7}{16}$ in. for gates 25 ft. square.

Gates over 25 ft. square shall be supported upon the bottom runners and provided with suitable hand-operating mechanism.

Where lifting gates are specified they shall be provided with suitable counter-balance weights, chains, pulleys and guides, and hand-operating gear.

Putty Glazing.

48. The glazing shall be of the quality and thickness specified in panes of the dimensions given. The edges and for $\frac{1}{2}$ in. in on both faces shall be painted with pure white-lead paint, and shall be dry before glazing. The sheets shall lap at least 2 in. over each other at the ends, and the laps shall generally be over the purlins. The glass shall be $\frac{1}{2}$ in. less in width than the centres of the astragals or glazing bars. Hardwood pins, $\frac{1}{4}$ in. in diameter and 1 in. long, shall be supplied and fixed through the astragals. A pin shall be placed at the ends of each bar, at each lap of the glazing, and in the centre of each sheet. Two zinc hooks or tingles, $\frac{1}{2}$ in. broad by No. 13 zinc gauge in thickness, shall be supplied and fixed in the putty fillet at each lap.

The tee astragals and putty fillets shall receive two coats of pure white-lead paint on the outside after glazing.

Note.—In the summer time the glass in the roofs may be painted with a mixture of white lead, driers and turpentine to obscure the glass. Sufficient white lead is used to thicken the driers and turpentine. If this mixture is put on in the beginning of summer, very little is left by the end, and is easily removed by washing with soda and water.

Patent Glazing.

49. Where patent glazing is specified, it shall be fixed in accordance with the specifications of the makers.

Corrugated Sheeting.

50. The side laps shall be one corrugation and the end laps at least 6 in. The side laps shall have bolts $\frac{1}{4}$ in. in diameter and spaced not more than 24 in. apart. The end laps shall have bolts $\frac{1}{4}$ in. in diameter spaced not more than 6 in. apart alternate pitch. Lead or other approved washers shall be placed under all heads or nuts of bolts on the outsides of sheets. All nuts and bolts shall be galvanised and placed in the ridges of the corrugations.

Where sheeting is used on the ventilators it shall be neatly curved to the radius shown on the drawings.

The sheeting shall be securely fixed to the purlins by hook bolts $\frac{5}{16}$ in. in diameter, or by galvanised straps $\frac{3}{4}$ in. in breadth by $\frac{1}{16}$ in. in thickness, or by galvanised clips $\frac{3}{4}$ in. in breadth by $\frac{1}{8}$ in. in thickness. The hook bolts, straps or clips shall not be more than 15 in. apart. The clips shall be fixed to the sheeting by No. 2 bolts $\frac{1}{4}$ in. in diameter.

The sheeting for side and end covering shall be fixed to the horizontal sheeting angles or girders by hook bolts $\frac{5}{16}$ in. in diameter, spaced not more than 15 in. apart. The horizontal blades of the sheeting angles shall be uppermost.

Galvanised corrugated sheeting, No. 16 W.G., shall have corrugations 5 in. centres and $1\frac{1}{4}$ in. in depth. The laps, bolts, etc. shall be arranged as for No. 18 W.G.

Where especially tight laps (joints) are specified, the edges of the sheets at the laps shall receive a coat of thick white lead, painted before they are brought together, or strips of canvas soaked in white lead may be inserted before closing up the sheets.

Galvanised Iron Flashings.

51. Galvanised iron flashings at ventilators and skews shall be made from plain sheets, No. 16 W.G. in thickness.

Galvanised Iron Ridges.

52. Galvanised iron ridge pieces shall be made from plain sheets No. 16 W.G. in thickness, and shall not be less than 18 in. in girth, and shall lap at least 6 in.

Where the roof is covered with corrugated iron the ridge piece shall be fixed to the corrugated iron by bolts $\frac{1}{4}$ in. in diameter, spaced not more than 10 in. apart.

Where the roof is covered by putty glazing, the ridge pieces shall be fixed to the tee astragals by $\frac{5}{16}$-in. diameter eye bolts. The space between the ridge piece and the glass shall be filled in by a wood strip $1\frac{1}{2}$ in. square, secured to the ridge piece by No. 2 wood screws.

Skew Timbers.

53. Skew timbers about 6 in. broad by 2 in. thick shall be secured to the purlins by bolts $\frac{1}{2}$ in. in diameter sunk into the timber. The galvanised skew sheeting shall be secured to the timbers by wood screws with raised heads, spaced about 12 in. apart alternately. Lead or other approved washers shall be used under the heads of all screws.

Slating.

54. Where slating is specified the slates shall preferably be 24 in. by 12 in., laid with a 3-in. lap, and securely fastened to the steel L purlins by No. 2 lead nails, $\frac{1}{8}$ in. square in each slate, bent round the purlin, or by two No. 14 gauge copper wires placed through each slate and secured round the purlin.

Double slates shall be put in at the eaves and ridges.

The slates shall be properly sorted and squared, and the vertical joint shall keep a true line from the eaves to the ridge.

Gutters and Downpipes.

55. The gutters shall be made from $\frac{1}{4}$-in. stamped steel plates in lengths of about 10 ft. The joints shall be made with a 4-in. by $\frac{1}{4}$-in. outer cover plate. A strip of canvas soaked in red lead shall be inserted between gutter and cover to make a thoroughly water-tight joint.

Valley gutters shall be at least 17 in. in width, which allows 10 in. between the glass or sheeting. Eave gutters shall be at least 12 in. in width. These dimensions allow the minimum space for a man to walk along the gutters.

The gutters shall preferably have a fall of 1 in. in 60 ft.

The down pipes shall be proportioned so that they shall have an area of at least ½ square inch for every 100 square feet of roof surface drained.

The down pipes shall be of cast iron and secured to the covering by suitable holdfasts. The joints shall be caulked with red lead and hemp.

Roof Gangways and Snow Boarding.

56. Gangways shall be at least 18 in. wide, and shall be made of 3 in. by 1 in. cross boards laid with 1 in. spaces upon 2 in. by 2 in. longitudinal runners. The runner on the roof slope shall have a 5 in. by 1 in. baffle board nailed to it and supported on the astragals.

The outer longitudinal runner shall be secured to a 2½ in. by 2½ in. by ¼ in. angle by ⅜ in. diameter bolts 2 ft. 6 in. pitch. The 2½ in. by 2½ in. by ¼ in. longitudinal runner shall be carried by 2½ in. by 2½ in. by ¼ in. angle standards spaced not more than 10 ft. apart.

On the eaves gangway a suitable handrail shall be fixed about 2 ft. 6 in. above the gangway level.

All gangways shall be made in convenient lengths to allow them being lifted for cleaning the gutters.

The gangways on roof slopes shall be formed of two sawn pine timbers 6 in. wide by 2 in. thick, and supported on the astragals at points not farther than 6 ft. apart.

Roof Ladders.

57. Roof ladders shall be about 12 in. in width, and be made of two sawn boards of red pine 6 in. wide by 1⅛ in. thick, laid with 1 in. space and supported upon the tee astragals by red-pine cross bars, 2 in. square by 5 ft. long, spaced about 7½ ft. apart. The treads shall be 1 in. square and about 12 in. apart. Two wheels, not less than 4 in. in diameter and 5 ft. centres, shall be provided at the bottom cross bar to run upon the baffle board of the roof gangways.

Brickwork.

58. Dwarf brick walls shall be generally 5 ft. high above floor line and 9 in. or 14 in. in thickness. The minimum thickness shall be used where no pressure from the side covering is carried by the wall.

Every course shall be bedded in mortar and thoroughly grouted with thin mortar until every joint is thoroughly filled before the next course is laid. Every fourth course of the walls shall be a header course. No joint shall exceed ⅜ in. in thickness.

Where the brickwork is built into steel framework the bricks shall be neatly cut and fitted into the steelwork, and the space between the steelwork and brickwork grouted up solidly. The brickwork shall be built solidly under the horizontal members of the framework, and grouted up through the holes provided in the steelwork. (See *Painting.*)

The width of the bottom of footings shall be at least twice the thickness of the wall diminished in regular offsets of not more than 2¼ in. The height from the bottom of the footings to the base of the wall shall be at least two-thirds of the thickness of the wall at its base. These dimensions shall not apply to dwarf walls.

The height of walls shall be measured as follows:—

1. Side walls. From the base of wall to the underside of the roof truss.
2. Gable walls. From the base of wall to half the height of the gable.

The height of a storey shall be measured as follows:—

1. Topmost storey. From the underside of floor beams to the underside of the tie of roof truss.
2. Other storeys. From the underside of floor joist to the underside of floor joist of the storey above.

In no case shall the height of a wall, if there are no floors, or the height of a storey, exceed fourteen times the thickness of the wall, where an external vertical load is carried, or sixteen times the thickness where no load is carried. The increased thickness may be provided for by piers properly distributed, so that their collective widths equal one-fourth part of the length of the wall.

In steel panelled construction the size of the panels shall not exceed 150 square feet in area, nor shall the thickness be less than one-twentieth of the maximum distance between the steel frames.

The thickness of cross walls shall be two-thirds the thickness of external walls, provided the openings do not exceed one half of the area of the wall.

No cross wall shall be less than $8\frac{1}{2}$ in. in thickness.

The length of a wall shall be measured between the centres of return walls or cross walls.

Bull-nosed bricks shall be used at all jambs of gates or doors.

Pointing.

59. Both sides of the brick walls shall be thoroughly pointed and hard drawn with a square key in perfectly level and perpendicular lines. The whole of the brickwork shall be pointed as the work proceeds.

Damp Course.

60. Every wall shall have a damp course, composed of materials impervious to moisture, extending throughout its whole thickness at a level of not less than 6 in. above the ground level. The damp course may be omitted in dwarf walls.

Timber Wall Plates.

61. Where timber covering is used above the dwarf wall, the wall plate shall be bedded in mortar on top of the wall and securely fixed to the wall by bolts $\frac{7}{8}$ in. in diameter and 18 in. long, with washers 4 in. by $\frac{1}{4}$ in. thick, built into the wall about 15 in. of its length, and not more than 3 ft. apart, or by wrought-iron straps, $2\frac{1}{4}$ in. by $\frac{1}{4}$ in. by 4 ft. long, bent over the wall plate and fixed through the wall by a bolt $\frac{3}{4}$ in. in diameter.

Foundations Under Walls.

62. The concrete foundations under walls shall be 6 in. wider than the width of the lowest course of footings, and the thickness shall be at least one-third of the width of the concrete to allow for any inequalities in the foundations liable to cause the concrete to break across through unequal settlement. In no case shall the concrete be less than 9 in. in thickness. Where necessary, it shall be reinforced with steel rods.

Foundations Under Columns.

63. The concrete foundations under the column bases shall be at least 6 in. wider all round than the base plate of the column, and the thickness shall be at least one-third of the maximum dimension of the concrete to allow for any inequalities in the foundations liable to cause the concrete to break across through unequal settlement.

The bottom of the column foundations shall not be nearer the surface of the ground than 3 ft., so as to be free from the effects of temperature.

Foundation Bolts.

64. In ordinary cases holes made with wooden boxes shall be left in the concrete for the foundation bolts. (The wooden boxes are removed when the concrete is set.) The holes shall be $4\frac{1}{2}$ in. square at the top. The bolts shall generally be $1\frac{1}{4}$ in. in diameter, with a square neck, and 2 ft. in length, with a washer on the bottom, 6 in. square by $\frac{3}{8}$ in. thickness. They shall be set in position to template and the concrete deposited round them, and shall be grouted up when the column is levelled and set.

One inch shall be allowed between the top of the concrete and the underside of the column base for grouting up when the columns are set in their correct positions.

Temperature.

65. No special provision shall be made for expansion or contraction in the length or width of the building if it is of steel framed construction throughout, except in the purlins of a glazed roof, where ample provision is made by bolting the joints about every 30 ft. Where the trusses rest on brick or masonry walls slotted holes shall be provided in the trusses in spans over 60 ft. and in the purlins and roof covering at the rate of $\frac{1}{2}$ in. for every 100 ft. in length.

Workmanship.

66. The whole of the workmanship shall be of a first-class character throughout and true to dimensions.

All sheared edges shall be planed or machined where stress is transmitted, and all holes in girders and columns and in thicknesses of $\frac{3}{8}$ in. and over shall be drilled.

Machine riveting shall be used wherever practicable, and all connections at the site shall be bolted.

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